ADDIS ABABA SCIENCE AND TECHNOLOGY UNIVERSITY

COLLEGE OF ARCHITECTURE AND CIVIL ENGINEERING

DESIGN AND ANALYSIS OF ASSYMETRIC SLIMFLOR BEAM WITH CROSS LAMINATED TIMBER COMPOSITE FLOOR

PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE IN STRUCTURAL ENGINEERING.

By

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Adisu Guta Garoma, declare that this thesis is my own original work that has not been presented and will not be presented by me to any other University for similar or any other degree award.

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Abbreviations

ASBs Asymmetric slimflor Beams

BS EN British standard European Norm

CLT/XLT Cross Laminated Timber

CO2 Carbon Di Oxide

CSA Canadian Standard Association

EBR Ethiopian Birr

EC0 Euro Code-0

EC1 Euro Code-1

EC5 Euro Code-5

ES EN Ethiopian standard European Norm

SCI Steel Construction Institute

FE Fire Engineered

SLS Serviceability Limit State

ULS Ultimate Limit State

WIS Wood Information sheet

Symbols

Latin Capital cases letter

A_{min} = Minimum bearing area

Ar Reduced section area

C_{Founds} Cost of foundations

C_{mat} Total cost of material

D Depth of the section

E Young's modulus of elasticity

$E_{d,SLS}$ Design Modulus of elasticity at serviceability limit state

E_{mean} Mean Modulus of elasticity

E_{steel} Steel Modulus of elasticity

E_{Timber} Timber Modulus of elasticity

G Shear modulus of elasticity for steel

H Warping constant.

I Second moment of area

I_{yc} Second moment of area of the top (compression) flange about its major axis

I_{yt} Second moment of area of the bottom (tension) flange about its major axis

J Torsional constant.

K_{def} Deformation Modification Value

k_{mod} Modification for load duration and moisture content

k_{sys} System strength factor

L_{cr} Critical buckling length

L_E Effective length for lateral torsional bucking

M_b Buckling resistance moment

$M_{R,flange}$ Bending resistance of the bottom flange

M_t Applied local bending moment per unit length of the bottom flange

Mx Maximum applied major axis moment

M_{yt} Resolved minor-axis bending arising from twist

N = Axial load from the column

S_x Plastic modulus of the section

V Shear force

$\mathbf{V_d}$ Design bearing force

W_{no} Normalized warping function at the compression flange tips

Z_f Reduced section modulus

Z_y Elastic modulus about the minor-axis

Latin Lower cases letter

b_c Breadth of the column

d_{min} = Minimum dimension of bearing area

$\boldsymbol{f}$ Characteristic bending strength of CLT.

f_1, f_2 & f_3 Component frequencies

f_b Characteristic bending strength

$\mathbf{f_{c,90,d}}$ Design compressive strength perpendicular to the grain

$f_{c,90,k}$ Characteristic compressive strength (perpendicular to grain)

$f_{m,k}$ Characteristic bending strength

$f_{v,d}$ Design shear strength

$f_{v,k}$ Characteristic shear strength

h_1 Distance from the center of the top flange to the center of gravity

h_b Distance from the center of gravity to the center of the bottom flange

m_{LT} Equivalent uniform moment factor for lateral torsional buckling

n Ratio of the Young's Moduli of the two materials in the composite section

p_y Yield stress of the steel.

r_y Radius of gyration about the minor axis

t Web thickness

t_{if} Minutes

y_o Distance between the center of gravity and the shear center

u Buckling parameter

Greek Lower cases letter

βn Notional char rate

γ_M Material partial safety factor (=1.25 for Glulam/CLT)

$\gamma_{M,fi}$ Partial safety factor for timber in fire

δ_{sw} Instantaneous deflection

λ_{LT} Slenderness ratio

ξ Joint detail between floor slabs

ϕ Maximum angle of twist

ϕ'' Second derivative of ϕ with respect to the distance from a support

σ_l Reduced available longitudinal bending strength in the bottom flange

σ_{ABP} = Allowable Bearing Pressure of the soil

σ_{byt} Bending stress in the flange tips

$\sigma_{c,0,k}$ Characteristic compressive strength parallel to the grain.

$\sigma_{c,90,k}$ Characteristic compressive strength perpendicular to the grain.

σ_w Warping stresses

τ Shear stress

Abstract

This thesis is angered to introduce the use of Cross Laminated Timber (XLT) floor Slabs in place of precast concrete units for low income peoples of urban and rural area of Ethiopia. As timber is a natural and renewable building resource, this replacement could bring an environmental and social benefit. It is also much lighter than concrete, meaning it may be possible to reduce the amount of steel used in the frame of a building. To be feasible, any structure built this way must adhere to the requirements of the Building Regulations and the structural design codes (Euro codes). The most critical aspects to meeting the requirements were anticipated to be ensuring adequate robustness and fire resistance.

In this thesis, the aforementioned requirements are explored and the ability of the hybrid System to perform was evaluated. The reuses of this timber steel composite floor with frames, constructability within a short period of time, cheapest when compared to concrete slab due to minimized foundation and frame size and cost because of light weight of timber, have a potential environmental benefits, Availability and easy accessibility of timber especially in rural area were investigated and a comparative study was performed to gauge the relative performances of the timber and concrete systems. The comparative study looked at how the amount of steel required in the frame, the loads to foundations, the cost, and the environmental impact changed when making the substitution for a range of situations. The loads to the foundations were reduced significantly in every case studied. The analysis for costs found that using timber would be a more expensive option, mostly due to the relative cost of the slabs themselves.

Finally, the advance for structural performance, physical property and cost analysis of timber-steel composite flooring over concrete floor will analyze for future Ethiopian construction industry.

CHAPTER ONE

1.1. Introduction

85% of the urban population of Ethiopia lives in inhuman, unhygienic and confined conditions. Their housing situation lacks infrastructure and is dominated by “chicka” type of construction (traditional construction method with mud and wood).The population growth of 2.8 % per year and the accelerated migration to urban centers (6 % and more per year) have dramatically increased the demand for affordable, decent housing. [1]

In Ethiopia today many buildings are done using reinforced concrete beams and slabs. The Reinforced concrete beams and slabs have large dead load and are costly especially for multi-storey buildings. Since Ethiopia’s economy is growing and development of infrastructure increasing, many multi-storey buildings are being constructed across the country.

The main challenge when designing for the foundations of these kinds of structures is the large dead load which mainly is as a result of the reinforced concrete beams and slabs. In areas with poor soils like black cotton soils, foundation design and construction is very expensive. [2]

Steel-timber hybrids can help reduce construction costs and also enable easy construction of Multi-stories on almost all Ethiopian soil types. Timber being much lighter than concrete, means it may be possible to reduce the amount of steel used in the frame of a building. Also by reducing concrete use will result to environmental benefits since research has shown that at least 5% of environmental pollution comes from cement production and use. On the other hand steel is lighter design wise and is more economical than RC in long clear span applications. For this reason steel beams are the best when doing commercial office multi-storeys. [2]

As a natural and renewable building material, timber has excellent ecological attributes. It acts as a carbon sink and has low embodied energy. The energy needed to convert trees into wood and hence into structural timber is significantly lower than that required by other structural materials such as steel and concrete With the increasing use of iron in buildings in the 19th century, timber was used in combination with metal for long span and heavily loaded structures. The development of mathematical procedures at this time for the design of structures and structural elements led to today's approach to engineering timber structures. The strength of sawn timber is a function of species, density, size and form of member, moisture content and duration of applied loading, together with strength-reducing characteristics such as slope of grain, knots, fissures and wane. Strength grading methods have been devised to classify timber using either visual strength grading or machine strength grading methods. [3]

The essence of the new approach is to use the Asymmetric slimflor beams (ASBs) with prefabricated massive timber slabs of cross-laminated timber (XLT) instead of reinforced concrete. The hypothesis is that a comparably performing timber slab will be significantly lighter than the equivalent concrete slab, allowing smaller, cheaper section sizes to be used in the building framework. As a result, the environmental impact of a timber floored building will be reduced as the embodied carbon of the steel frame will reduce with the section sizes, and the significant embodied carbon in the concrete is removed altogether. Concurrently, the cost of the structure will fall, with reduced framework and foundation costs, and quicker rates of construction. Another potential advantage, both economically and environmentally, is that buildings using this technique could be dismantled at the end of the building's working life, and the slabs (and even frame sections) reused rather than recycled. If such as system can be proven to be feasible, it will be both a financially and environmentally attractive option for steel building design of the future. [4]

To include Cross Laminated Timber (CLT) in the Ethiopian Building Code standard, research must be done on the Bending Strength, Shear Strength and Rolling Shear Strength of the material depending on the number of panels. These values would be required in order to perform the various checks necessary to determine the capacity of a CLT cross-section. For load perpendicular to the plane, a more accurate model of CLT elements is required to be done using composite theory. This model should be able to take into account the effects of rolling shear in every other layer. The strength and stiffness of each layer in the direction of loading is modeled separately, and compliance & stiffness matrices are used to relate the behavior of each layer in the laminate in order to obtain the stress and stiffness variation throughout the cross-section.

This paper will attempt to analyze and design the structural performance and cost analysis of timbers-steel composite flooring for future Ethiopian Construction industry.

1.2. Statement of the problem

Analyze the structural performance and cost analysis of timber-steel composite flooring for future Ethiopian Construction sector.

1.3. Objective of the study

1.3.1. General objective

- ✓ To Analyze & Design the structural performance of Asymmetric slimflor beams with Cross laminated Timber composite floor.

1.3.2. Specific objectives

- ✓ To drive the common design criteria of Cross laminated timber with Asymmetric slim-floor beams composite flooring formulas.
- ✓ To evaluate the cost of timber-steel composite flooring with concrete floor slabs Based on current Ethiopian construction market.
- ✓ To Evaluate Material performance of timber-steel composite flooring for what it will be designed.

1.4. The Study Design and Methodology

Now a days mainly the building floor in Ethiopia is concrete solid slab, concrete precast slab unit or Ribbed slab, which results, high carbon emission, poor quality of construction, un able to reuse, coasty and unavailability of quality material are its major disadvantages.

These mentioned ideas found in different books & journals will be selected and analyzed to find best timber-steel composite floor structure which gives maximum structural performance with low over all weigh of the building cost especially for low income housing project in Ethiopia.

1.5.Literature review:

A review of all existing relevant literatures, thesis document, Journals and Internet resources in the structurally replacing concrete floor slabs with timber-steel in composite construction Structural timber design to Euro code 5, Design of Asymmetric slim floor Beams with precast concrete slabs, cross laminated Timber, Behavior of composite slim-floor structures in fire, Sustainable low cost housing in Ethiopia, Timber-Steel hybrid slab, structural implications of replacing concrete floor slabs with timber in composite construction and method of construction used.

1.6. Significance of the Study

These are few of the significance of Timber-steel composite floor slab.

- Reduce the overall weight of the entire building and minimize foundation size as well as able to construct the foundation on every Ethiopian soil types.
- Reduce steel usage in entire structure of the building.
- Due to the availability and ability of the timber & steel reuse it is recommended especially for low cost housing program and highly growth of urbanization in the country.
- Simplicity of construction (easier to install the assembled floor sections.)
- It can be used as floor finish tile of the building in addition to its structural use.
- Lighter construction than a current low-cost concrete building
- Ease of transportation and installation
- With low cost and no carbon emission it is recommended for the low income households and good environmental simulation.
- It will be a new design approach to architects and structural engineers about configuration of floors in Ethiopia.
- Ministry of Urban Development and Construction, AAHDPO, Consulting and Construction firms can be benefited from the output of this research work in terms of low carbon emission, constructability, reuse, fire resistance, Robustness, Low cost and availability of material quality and time wise.
  - Now days the biggest environmental issue of the worlds is deforestation and high carbon emission, the comparative advantage of this research may help the country to grow the tree until it is able to use for timbering.

1.7. Scope of the Study

This study is mainly aim to apply the steel-timber hybrid floors particularly for Ethiopian Construction sector and is further investigate on hybrid limit state design of steel-Timber composite flooring specially the ultimate limit state design to make the issue practical. In addition the comparison of Cross Laminated Timber and Asymmetric slimflor beam flooring with currently famous solid concrete floor constructed in entire country specially in terms of cost, reuse of material, ease of construction, environmental protection while production and construction stage of embodied energy (CO2), the total weight of structural steel in entire building and total weigh of building to reduce foundation size of the two types of floor were addressed. However it needs further investigation.

The study is mainly depends on Euro code's, Ethiopian Building Code Standard new version, British standards and different journals which are fully arrowed their references to those mentioned codes.

1.8. Limitation of the Study

It is limited to figurative analysis and design of Cross Laminated Timber and Asymmetric slimflor beam flooring rather than conducting material strength laboratory test, design and cost analysis of connection between ASB & CLT and use of design soft wares.

CHAPTER TWO

2.1 Literature review

2.1.1. Cross Laminated Timber

Timber is a natural building product, formed from the cutting down and machining of trees. Its mechanical properties vary with species, moisture content and time. However, it is also very strong for its weight, durable, and insulating against heat and sound. When sourced from managed forests, it is also a sustainable, renewable material resource. Engineered Wood Products such as Glue Laminated Timber (GluLam), laminated veneer lumber (LVL) and cross-laminated timber (XLT) now allows more predictable behavior and more uses for timber products.

Cross-laminated timber slabs are formed from multiple layers of timber, glued together with each successive layer oriented so the grain direction is perpendicular to the one previous. As a result, the effect of imperfections in the timber such as knots are minimized and balanced out, producing a more uniformly behaving element. Currently, these slabs are produced in Germany, Austria and Switzerland, and can span up to 8m. Slab thicknesses can be produced up to 500mm thick if necessary. These panels can be used to form floors and load bearing walls of a structure, for example Bridport House in Hackney, London. Whilst structures made purely from cross-laminated timber are currently only economically viable to a height of about 8 storeys, recent studies have shown that combining cross laminated timber with a concrete core or steel support beams, structures of up to 40 storeys are possible. Notable manufacturers of the slab are Massivholz KLH, Binderholz BBS, and MMK, all originating from Austria. [4]

2.1.2 Eurocode Methods

Timber's immensely variable behavior means Eurocode 5, the design code for timber structures, incorporates many strength reduction factors to account for the duration of loading (since timber weakens under prolonged loading), the moisture content of the element, the size of the element, and the ability of multiple members to act together. In addition, the anisotropy of timber means a given species of wood has many more strengths than steel or concrete, relating to the direction and type of loading.

As a natural material, both between species and within populations there is great variation in the values of these strengths, meaning design is much more uncertain. Manufactured wood products have slightly more consistent behavior, but are still subject to safety factors relating to this uncertainty. [4]

Since Cross-Laminated Timber is a relatively new product, it does not have its own set of material and modification factors in the Eurocode. Hence, in the course of this study, when designing to Eurocode requirements, factors relating to Glu-Lam are used as it is the most similar wood-product to cross-laminated timber categorized in EC5.[4]

2.1.3 Service classes

(1) P Structures shall be assigned to one of the service classes given below:

NOTE 1: The service class system is mainly aimed at assigning strength values and for calculating deformations under defined environmental conditions.

NOTE 2: Information on the assignment of structures to service classes given in (2) P, (3) P and (4) P is given in the National annex.

(2)P **Service class 1** is characterized by a moisture content in the materials corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeding 65 % for a few weeks per year.

NOTE: In service class 1 the average moisture content in most softwood will not exceed 12 %.

(3)P **Service class 2** is characterized by moisture content in the materials corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeding 85 % for a few weeks per year.

NOTE: In service class 2 the average moisture content in most softwood will not exceed 20 %.

(4)P **Service class 3** is characterized by climatic conditions leading to higher moisture contents than in service class 2.

Layers of softwood glued together (Figure 2.1). The gluing is along the full surface of each panel. Panels are usually manufactured with their outer layers oriented in the direction that the CLT is going to span as shown in Figure 2.2.

Figure 2.1 Cross-Laminated Timbers

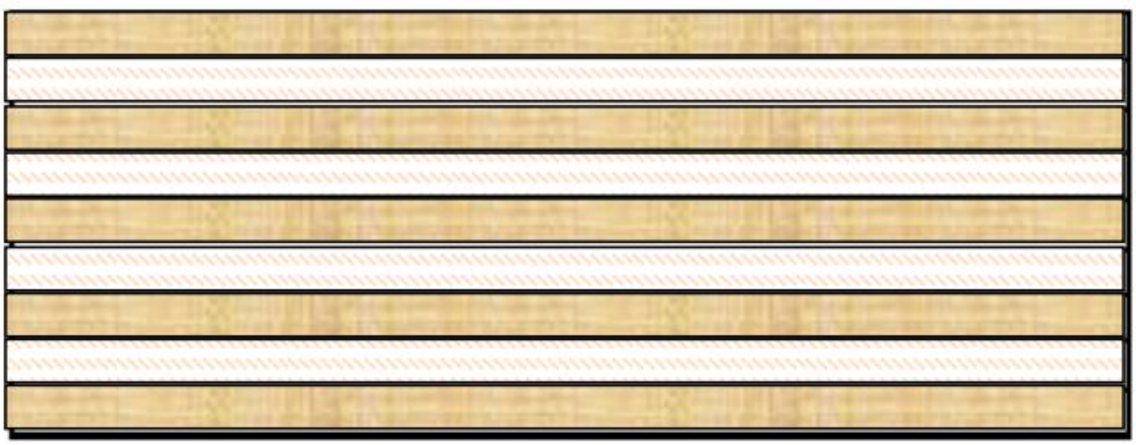

Figure 2.2 Orientations of the Outer Layers

2.1.4. BACKGROUND ON CROSS-LAMINATED TIMBER

Typical material properties of CLT depend on the type of wood that it is made of. The CLT material properties, life cycle, and production of CLT are described in the following sections.

2.1.5. *Typical Material Properties of CLT*

Material properties of CLT vary according to the manufacturer and basic materials. Wood that is used in production of CLT is normally spruce, but larch and pine may be available.

The type of wood that is used for manufacturing of the CLT affects the strength of CLT in bending and shear. The common strength grades (WIS 4-7) for the laminates are in the range C16 to C24 and at least one manufacturer offers 'glulam' grades (BS EN 1194) GL24H to GL28H.

Moisture content generally has a significant effect on wood performance due to shrinkage. Moisture content at delivery for CLT is typically 8-14%. Designers will find that working stresses are low due to the large cross section.

Surface quality of CLT is important due to both architectural features of it, and structural use. Classification of the surface quality of the panels is as in following:

- ✓ Grade (or non-visible quality) surface is suitable for lining
- ✓ Interior Grade (or residential visible) surface is suitable for exposed residential internal structure
- ✓ Some manufacturers have a third grade, Interior Grade (or Industrial visible) surface that is suitable for exposed industrial internal structure

CLT is generally manufactured with the standard properties as shown in Table 2.1.

Table 2.1 Typical CLT Properties

Width:	up to 2,95 m
Length:	up to 16 m
Thickness:	19 mm, 27,5 mm, 35 mm and 42 mm
Pre-cutting:	Any cuts for windows, doors, and so on
Wood types:	Spruce (Pine and Larch) on requesst
Grading:	C 24 / C 16 (in line with DIN 4074) higher grades on request
Moisture content:	12% +/- 2%
Bonding adhesive:	Formaldehydfree adhesive for edge bonding, finger jointing and surface bonding
Optical qualities:	Standard and visible quality
Surface finish:	Sanded

CLT is manufactured in 3, 5, 7 and multilayer composites with different component Thicknesses. Fundamentally, it is possible to produce any overall thickness by combining the following layers: 19 mm, 27, 5mm, 35 mm and 42mm.

Maximum overall thickness: 400 mm. It is also possible to apply engineered timber products such as wood-based panels and plywood to CLT by surface bonding.

2.1.6. CLT Life cycle

The life cycle of CLT according to Gerhard Schickhofer's presentation at UBC, regarding CLT and the Austrian Practice, started from Europe in 1993. In 1993, the first CLT residential building was built in Austria. In 1995, the first multi- storey building was built, and in 1998 the 1st multi-storey building was built using CLT which was approved by

Austria's national building code. In 2008, the Austrian code was made which is under use nowadays.

2.1.7. Production Process of CLT

The first step for production of CLT is from the cross cutting the logs as shown below in Figure 2.3.

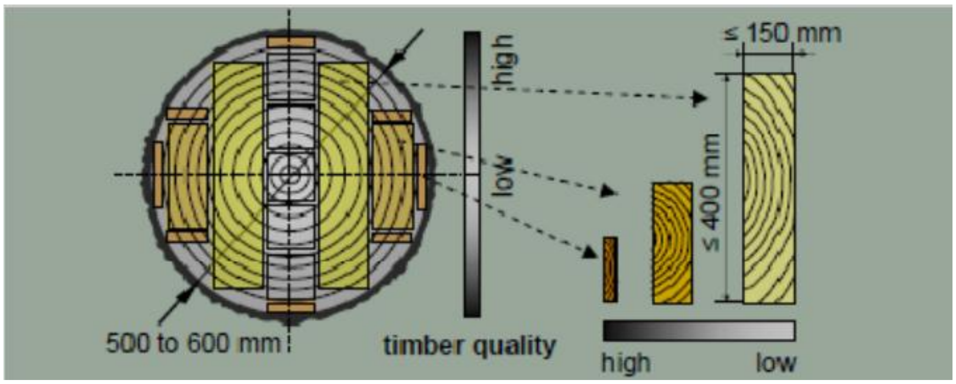

Figure 2.3 Cross Cut Log

Then the process of grading and trimming of the boards is done to exclude the parts of the wood that have flaws such as knots, and checks. In this part the long members are formed up to 16 m of length.

The next step is to make the panels from the finger jointed lamella. Finger jointing is a method of connecting timber and wood members to make a continuous member. Finger joints are made of a set of complementary rectangular cuts in two or more pieces of wood which are locked and glued together as shown in Figure 2.4

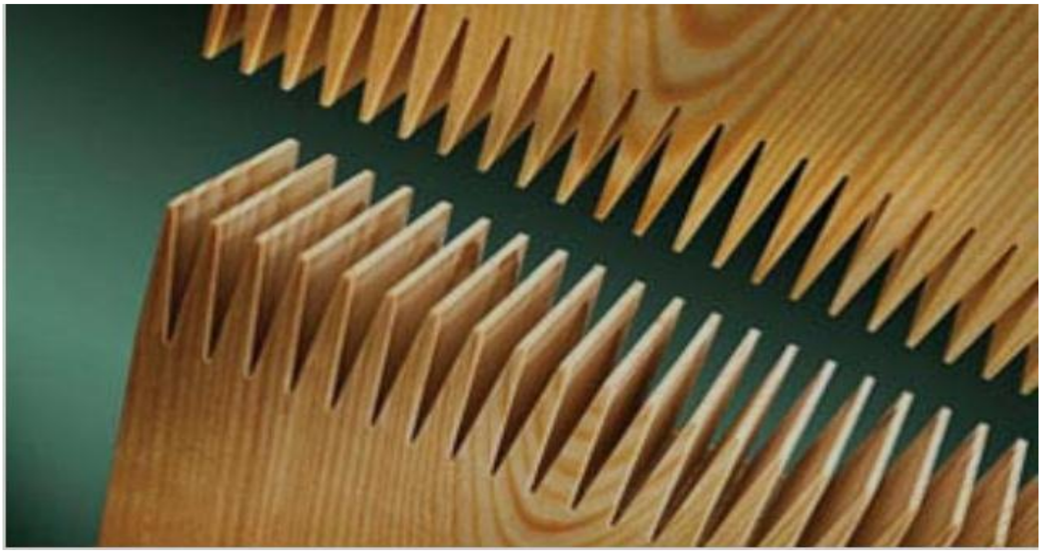

Figure 2.4 Precision Finger Joint Cutting

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After making the finger jointed board members, they are edge glued to each other to make panels. The panels are also connected to each other using adhesives in perpendicular directions to each other as shown in Figure 2.5.

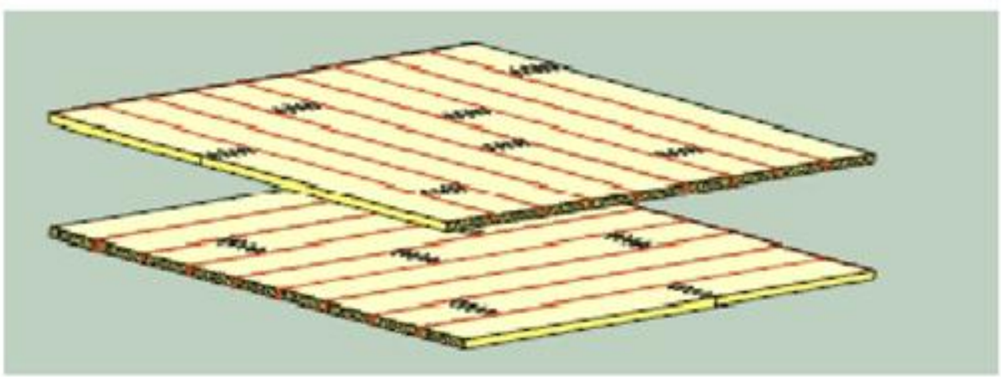
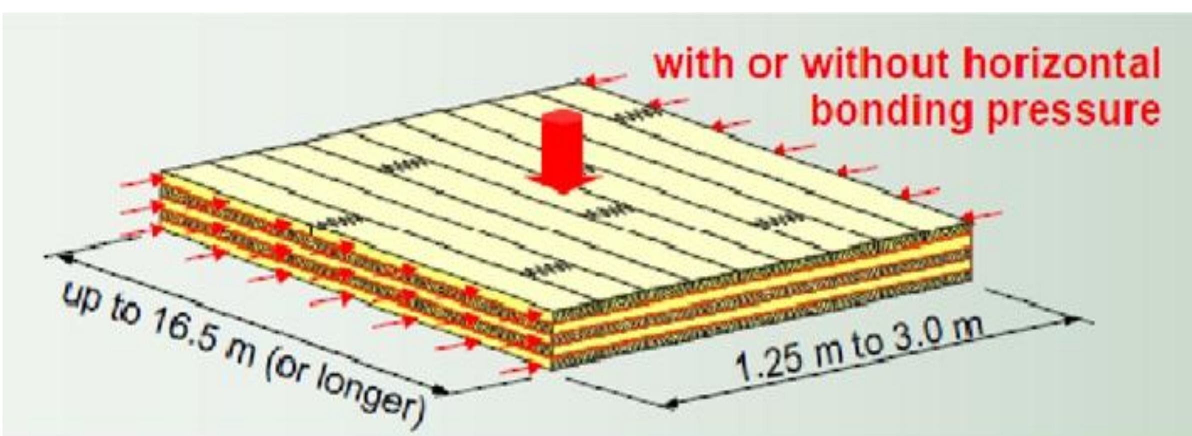


Figure 2.5 Edge Gluing of Panels

To attach the panel layers together; the face gluing is done, also a horizontal bonding pressure up to **0.6MPa** by hydraulic equipment is used to make increase



the strength of panels. In Figure 2.6 a 4 layered CLT panel is illustrated.

Figure 2.7 shows a completed 5 layered CLT panel. In the diagram, the figure joints, the thickness, and the width and length of a typical section are shown.

Figure 2.6 Face Gluing and Horizontal Bonding Pressure

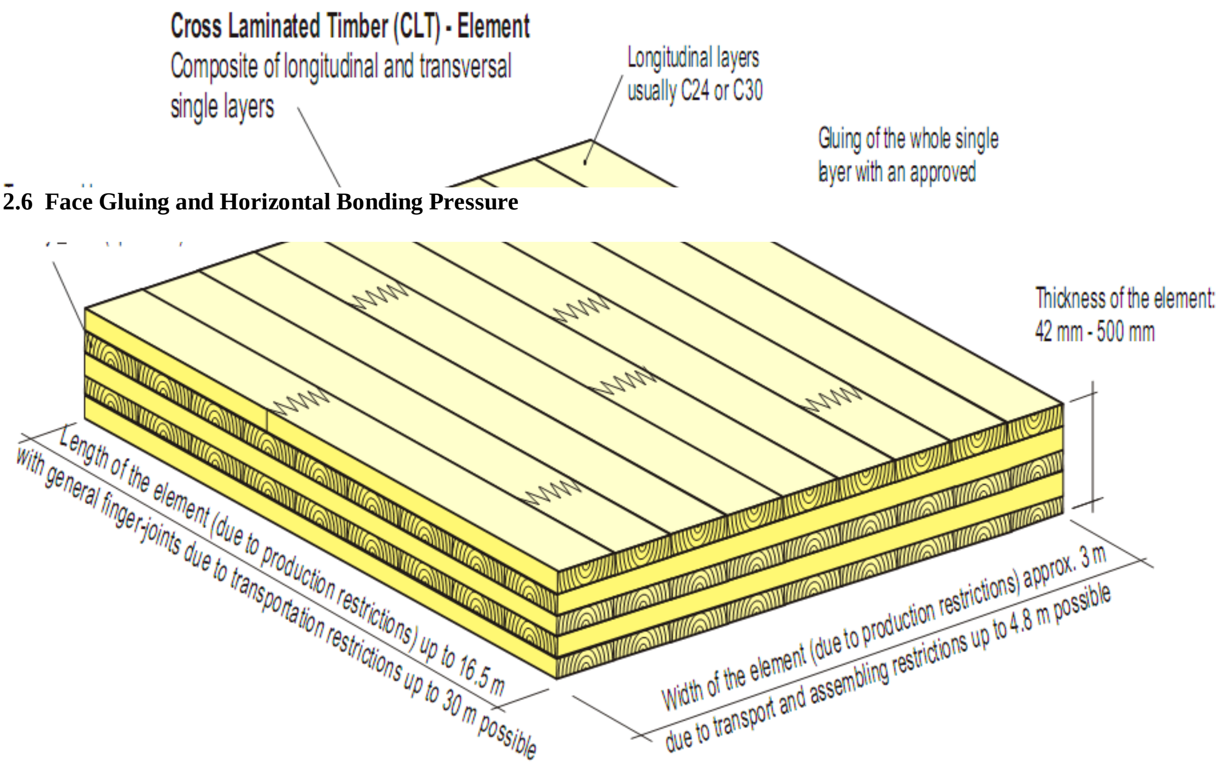


Figure 2.7 CLT Panels Using Finger Joint Connections

2.2. APPLICATION OF CLT IN STRUCTURS

In this part, application of CLT, advantages and disadvantages of the material are described.

2.2.1. *Advantages and Disadvantages of CLT*

A. CLT has several structural benefits such as

- ✓ Having a very high axial load capacity due to its large bearing area.
- ✓ High shear strength to resist the horizontal loads.
- ✓ The dead weight of CLT (used in walls of buildings) also reduces the need for mechanical holding down to resist the overturning forces.

- ✓ For CLT panels that are used in walls, one advantage is that the buckling in the plane of the wall is unlikely except for the isolated columns and piers.
- ✓ Being used in building structures is that it can be used for shallow floors, and structural fixings are easy to provide.
- ✓ It contains very few defects and flaws because of its process of production. During the second process of making the lamella elements, the flawed sections are cut out so a continuous panel can be formed.
- ✓ Is comprised of bonded, cross-laminated single layered panels. This ensures construction-relevant benefits and problem-free connections. Due to its non-revealing connection details (between the panels) it provides architects with an aesthetically appealing material to work with. The lack of joints results in better hermetic sealing, heat transmission, and water vapor diffusion, acoustic and fireproofing properties.
- ✓ Formaldehyde-free and environmentally friendly adhesives are employed for bonding. The cross structure of CLT components guarantees integral stability.

B. Disadvantages of CLT

- ✓ The significant dead weight and thickness of cross section, as well as difficulty in producing architecturally interesting shapes, since CLT panels are difficult to curve or bend. Care also needs to be taken in analyzing the service conditions of the CLT panels and ensuring that moisture is not a problem during construction and operation.
- ✓ There will be some difficulties in applying design code calculations for CLT, since the interaction between individual wood lamina in a CLT panel is relatively complex and would require extensive testing and analysis before simplified code equations become available.

2.2.2. ***Structural Use of CLT***

CLT has different applications in Europe, especially in Austria and Germany. It is majorly used as wall assemblies and one and multi-storey buildings such as schools, residential houses and storage buildings Like Figure 2.8 10 Storey Timber Building, Mail borne, Australia 2013.

Figure 2.8 10 Story Timber Building, Mel bourne, Australia 2013 [http://victoriaharbour.com.au]

One other application of CLT is for decking of bridges. The main structures of bridges are commonly made of steel or timber, but the decking material is CLT. One good example of CLT bridges is the Wandritsch Road Bridge in the city of Murau, in the state of Styria, Austria. This bridge which was built in 1998 has CLT as its decking material. This bridge uses a 9 layers CLT as its decking material shown in Figure 2.9.

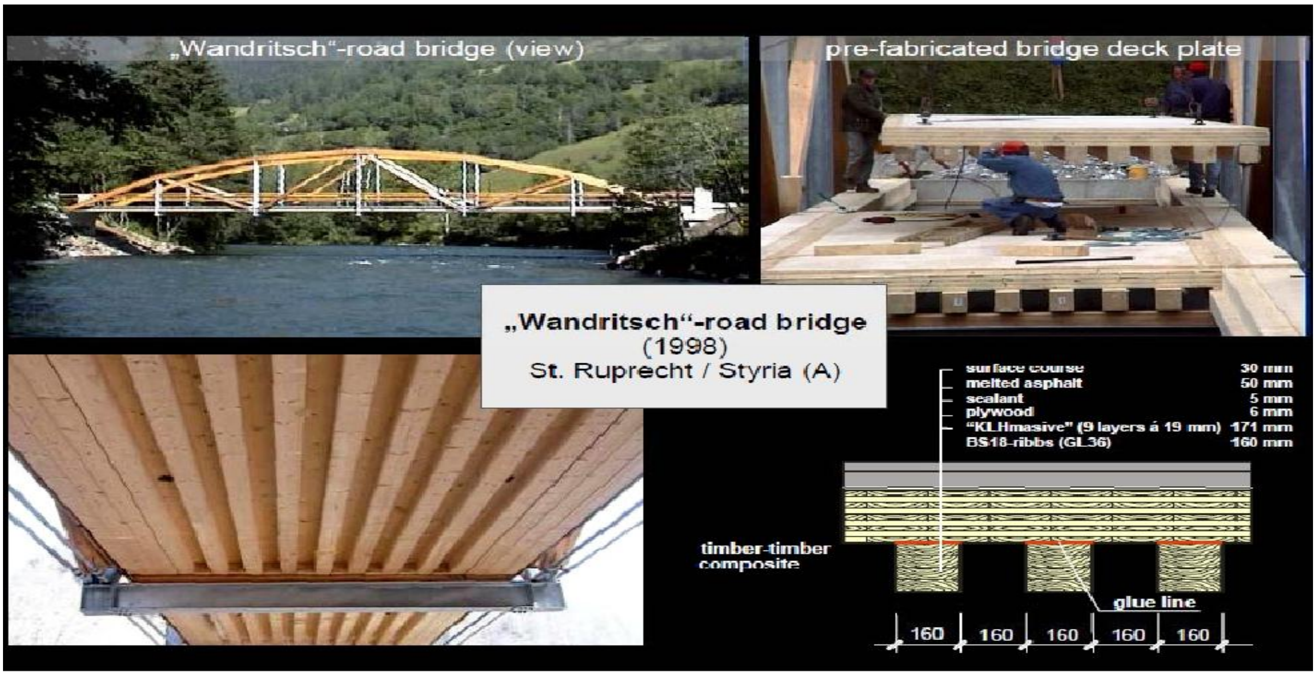


Figure 2.9 CLT Used as Decking in Wandritsch Road Bridge [5]

Another CLT bridge that is shown in figure 2.10 is constructed in Feldbach, Styria, Austria. This bridge is a pedestrian bridge which its decking is made of CLT.

Figure 2.10 Raabsteg Feld Bach CLT Bridge [5]

2.3. Asymmetric Slimflor Beams

The Asymmetric Slimflor Beam (ASB) is produced by hot rolling an asymmetric I-section, where the bottom flange is 110 mm wider than the top flange. This results in a ratio of the area of the top flange to the bottom flange of approximately 60% (Figure 2.11). The ASB was originally developed for use with deep decking in *Slimdek* construction but it may be used with PC Units, provided that certain design principles and geometric limits are observed.

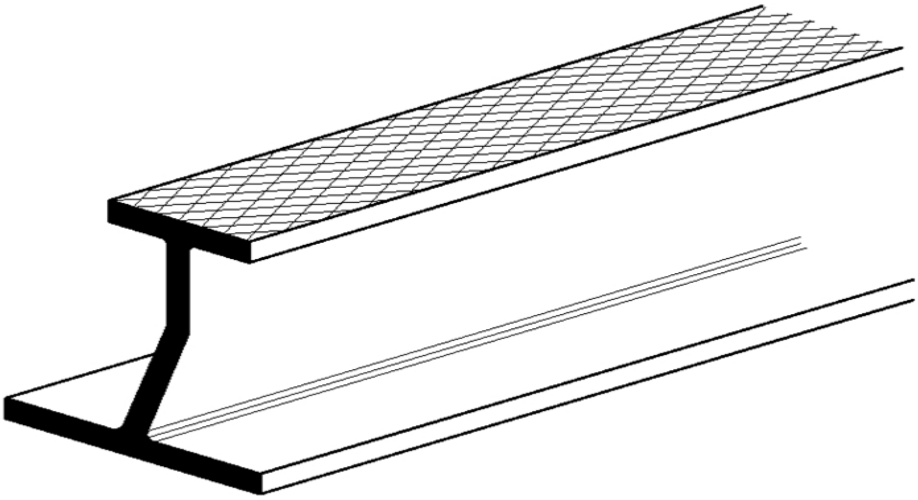


Figure 2.11 Typical Asymmetric Slimflor Beam

Two types of ASB sections are available: ASB, and ASB (FE), (Figure 2.12). ASB sections have thin webs and the exposed bottom flange may require fire protection to achieve more than **30** minutes fire resistance. ASB (FE) sections are engineered for optimum characteristics in the normal and fire conditions; these beams have a thicker web in order to achieve a fire resistance up to **60** minutes when unprotected, provided that there is a continuous encasement of the web throughout the length of the beam.

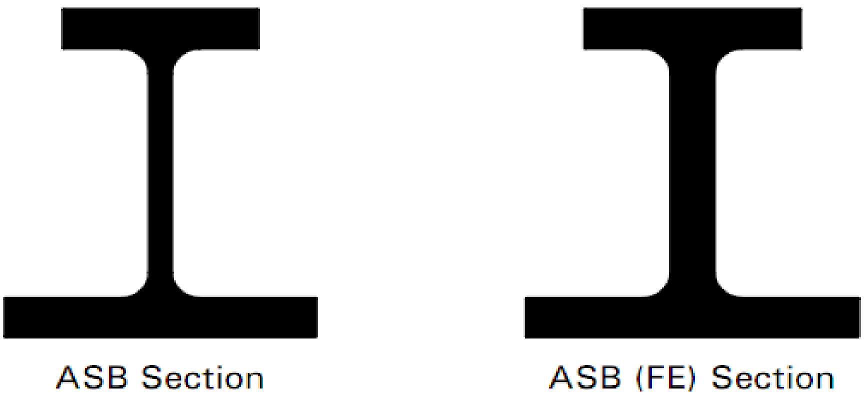


Figure 2.12 Types of Asymmetric Slimflor Beam

Dimensions of the ASB section range are shown in Table 2.3 and in Figure 2.13. The structural properties are shown in Table 2.2

Table 2.2 ASB dimensions for detailing

Designation	Mass	Depth	Width of Flange		Thickness		Root Radius	Depth between Flanges
			Top	Bottom	Web	Flange		
		D	B_t	B_b	t	T_t & T_b	r	d_1
	kg/m	mm	mm	mm	mm	mm	mm	mm
300 ASB(FE)249	249	342	203	313	40	40	27	262
300 ASB196	196	342	183	293	20	40	27	262
300 ASB(FE)185	185	320	195	305	32	29	27	262
300 ASB155	155	326	179	289	16	32	27	262
300 ASB(FE)153	153	310	190	300	27	24	27	262
280 ASB(FE)136	136	288	190	300	25	22	24	244
280 ASB124	124	296	178	288	13	26	24	244
280 ASB105	105	288	176	286	11	22	24	244
280 ASB(FE)100	100	276	184	294	19	16	24	244
280 ASB74	73.6	272	175	285	10	14	24	244

Note: ASB(FE) are fire engineered sections

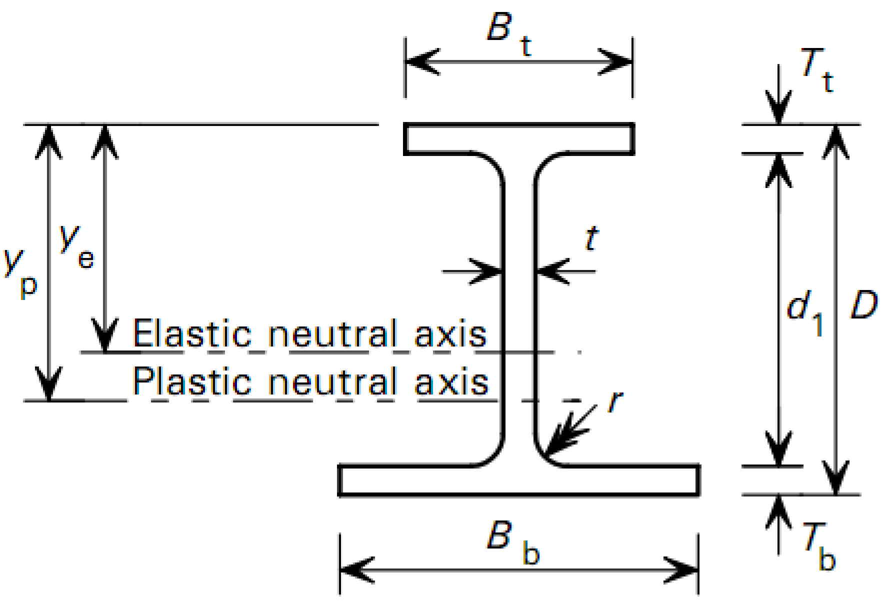


Figure 2.13 Identification of ASB dimensions in Table 1

Table 2.3 ASB properties

		Second moment of area		Radius of gyration		Elastic neutral axis	Elastic modulus		Plastic neutral axis	Plastic modulus		Buckling parameter		Torsional index	Warping constant	Torsional constant	Area of section
	Mass	I_x	I_y	r_x	r_y	y_e	Z_x Top	Z_x bottom	Z_y	y_p	S_x	S_y	u	x	H	J	A
	kg/m	cm^4	cm^4	cm	cm	cm	cm^3	cm^3	cm^3	cm	cm^3	cm^3	–	–	dm^6	cm^4	cm^2
300 ASB(FE)249	249	52900	13200	12.9	6.40	19.2	2760	3530	843	22.6	3760	1510	0.82	6.80	2.00	2000	318
300 ASB196	196	45900	10500	13.6	6.48	19.8	2320	3180	714	28.1	3060	1230	0.84	7.86	1.50	1180	249
300 ASB(FE)185	185	35700	8750	12.3	6.10	18.0	1980	2540	574	21.0	2660	1030	0.82	8.56	1.20	871	235
300 ASB155	155	34500	7990	13.2	6.35	18.9	1830	2520	553	27.3	2360	950	0.84	9.40	1.07	620	198
300 ASB(FE)153	153	28400	6840	12.1	5.93	17.4	1630	2090	456	20.4	2160	817	0.82	9.97	0.895	513	195
280 ASB(FE)136	136	22200	6260	11.3	6.00	16.3	1370	1770	417	19.2	1810	741	0.81	10.2	0.710	379	174
280 ASB124	124	23500	6410	12.2	6.37	17.3	1360	1900	445	25.7	1730	761	0.83	10.5	0.721	332	158
280 ASB105	105	19200	5300	12.0	6.30	16.8	1150	1610	370	25.3	1440	633	0.83	12.1	0.574	207	133
280 ASB(FE)100	100	15500	4250	11.0	5.76	15.6	995	1290	289	18.4	1290	511	0.81	13.2	0.451	160	128
280 ASB74	73.6	12200	3330	11.4	5.96	15.7	776	1060	234	21.3	978	403	0.83	16.7	0.338	72	93.7

2.4. PRACTICAL CONSIDERATIONS

2.4.1. Initial Consideration

- Assumptions
    - For erection and joint with ASB purpose the precast concrete unit property is also taken for Cross laminated timber units. Hence in this paper CLT Unit replaced by PC unit.

Prior to detailed design, the following practical issues should be considered by the designer wishing to use Asymmetric *Slimflor* Beams and cross laminated Timber:

- Nominal bearing length (on the ASB flange) and tolerances for erection.
- Geometric limitations and end preparations of CLT Units.
- Type/detailing of edge beams.
- Temporary stability of the ASB during installation of the CLT Units.

Other important initial considerations covered in later sections in this publication
Include:

- 'Robustness' (against explosions, etc).
- Use of the floor as a diaphragm to transfer in-plane wind forces.
- Fire resistance.
- ASB-to-column connection detailing.

2.4.2. Nominal bearing lengths and tolerances

The nominal bearing length can be determined by consideration of the:

- Practicality of erection.
- Risk of fall-through during erection.
- Minimum bearing length.
- Positional and length tolerances.

2.4.3. Practicality of erection of the CLT units.

Notch / chamfer when necessary

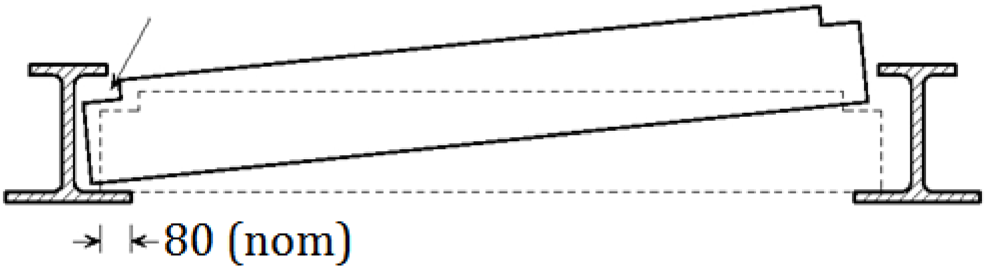


Figure 2.14 Erection of CLT units

The specified length of the CLT units is related to the nominal bearing length. A parametric study by the SCI has shown that, by consideration of the positional and length tolerances in the range of spacing's between ASBs of 1.5 to 11.0 m, a practical nominal bearing length is 80 mm. To facilitate installation, the depth of the unit must be either shallow enough, or have sufficient end notch or chamfer, to avoid clashing with the top flange of the ASB (see Figure 2.14). The notch or chamfer should meet the geometric requirements set out in Section 2.5. The unit is installed, initially on a slope, and maneuvered on to one ASB before sliding back to the other, to equalize the bearing length.

2.4.4. Risk of 'fall-through' of the CLT units

The CLT units have to slide between beams for final positioning. It is obviously essential that there is little risk of the unit falling through between the ASBs at this stage, even though the unit should be continually supported from the lifting equipment during this process. If a nominal bearing length of 80 mm is specified, there is minimal risk of this occurring for the commonly accepted tolerances referred to below. Particular care is necessary in situations where the CLT units is supported on an ASB at one end and on a down stand beam at the other, such as may occur at an edge beam. Again, an 80 mm nominal bearing length is recommended for the ASB and at least that at the down stand beam support.

2.4.5. ***Minimum bearing length***

Once the CLT units is in its final position, the actual bearing length measured on site should not be less than 40 mm, as recommended in BS 8110-1.This value is used for design of ASB sections with CLT units, rather than the nominal (80 mm) value, because it induces more torsion in the ASB section and is therefore conservative.

2.4.6. ***Tolerances***

The tolerances affecting the relative positions of the ASB and the CLT units are:

- Steelwork dimensional tolerances (as erected).

- Tolerances on length of the CLT units.
- Tolerance of placement of the CLT units.

A full consideration of these tolerances is given in SCI Publication P287 [11], but values assumed here are, as follows:

Steelwork dimensions (beam to beam)

CLT unit's length	± 10 mm
	± 12 mm for units < 6 m long
CLT unit's positions	± 18 mm for units ≥ 6 m long
	± 10 mm for units < 6 m long

± 15 mm for units ≥ 6 m long

2.5. Geometric limitations of CLT Units

CLT Units are square-ended, notched or chamfered. When they are installed, the clearance between the tip of the top flange and the nearest part of the precast unit should be **60 mm,** in order to permit installation of the units and proper placement and installation of the Binding material around the ASB. Typical end preparations based on a review of units currently available are shown in Figure 2.15 and Figure 2.16, where dimension d_{HC} is the overall depth of the CLT Unit.

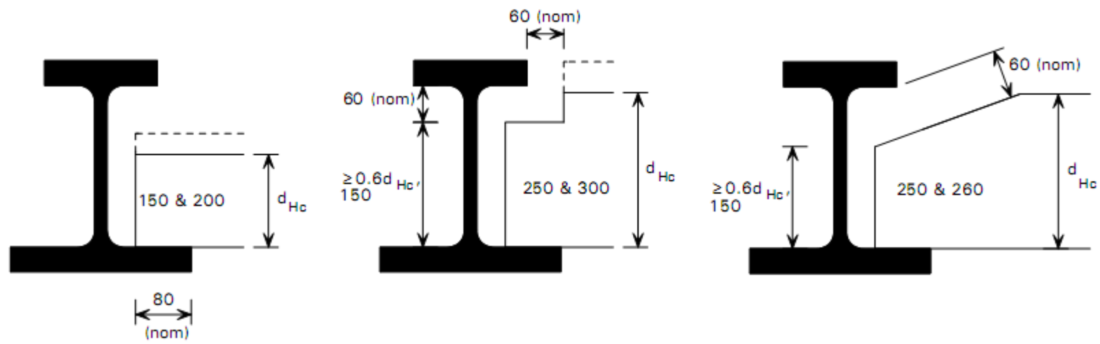


Figure 2.15 CLT Unit end details with 280ASB sections

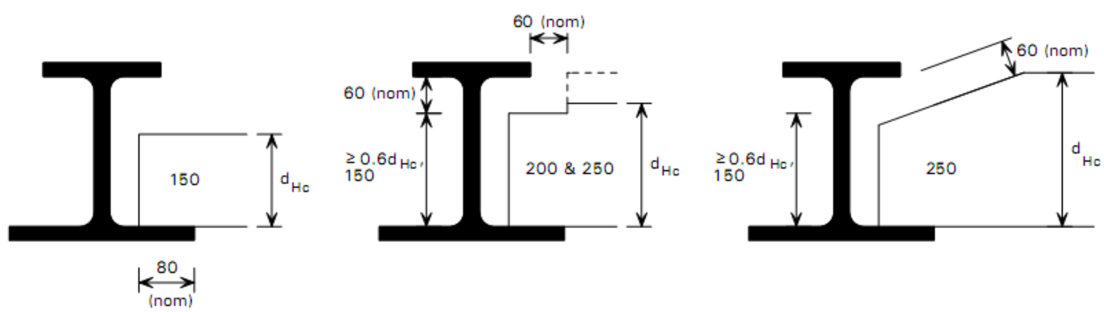


Figure 2.16 CLT Unit end details with 300ASB sections

CHAPTER THREE

3.1. Design of Cross Laminated Timber floor

3.1.1. CONCEPT OF LIMIT STATE DESIGN

The concept of limit states design is that for defined states a structure may be classified as either satisfactory or unsatisfactory. The limit state being the state beyond which the structure will no longer satisfy the design criteria and will be classed as unsatisfactory. It is possible to establish limit states to define limits of satisfaction for numerous issues, and to simplify the design process EC0 has defined two such states: 56 Structural Timber Design to Eurocode 5 ultimate limit states – associated with forms of structural failure/collapse, serviceability, e.g. states deflection and vibration conditions. For the ULS one is dealing with extreme safety conditions and for the SLS it is the level of comfort and appearance that is being addressed and, the level of reliability used in the design process will be different for each of the states. Design situations (EC0, 3.2) The structure must be designed for the effect of actions and environmental factors that will occur during the design working life and EC0 defines four design situations that must be considered:[2]

1) Persistent design situations: conditions of normal use i.e. self-weight and imposed loading, including wind, snow, etc.
2) Transient design situations: refers to temporary conditions, e.g. during construction or repair.
3) Accidental design situations: exceptional conditions, e.g. explosion or impact.
4) Seismic: conditions arising from a seismic event.

3.1.1.1. Ultimate limit states

These are the limit states that concern the safety of people and/or of the structure. Also protection of the contents supported by the structure can be included for in these states provided this requirement is agreed with the designer.

Specific attention is drawn to the following ULS that are relevant to timber structures and must be considered:

- ✓ Loss of equilibrium of part or all of the structure
- ✓ Failure by excessive deformation
- ✓ Failure as a mechanism

- ✓ Failure due to rupture
- ✓ Failure due to loss of stability.

3.1.1.2. Serviceability limit states

These are the limit states that concern the functioning and appearance (excessive displacement and cracking/distress) of the structure as well as the comfort of the users.

The combinations of actions associated with these types of SLS are as follows:

a) No exceedance permitted – the characteristic combination is to be used.

b) Frequency and duration of exceedance events agreed – the frequent combination is to be used.

c) Long-term exceedance events agreed – the quasi-permanent combination is to be used.

The criteria used for the verification of the SLS should be associated with the following matters:

i. Deformations that affect appearance, that causes damage to finishes or non-structural members that affect user comfort and the functioning of the structure.

ii. Vibrations that cause discomfort to users or limit the functionality of the structure.

iii. Damage that adversely affects appearance, durability or the functionality of the structure.

3.2.Limit states design

In limit states design, structural and load models are set up for each limit state and the design is verified by demonstrating that none of the states will be exceeded when design values of actions, material or product properties and geometry are used in the models.

The following symbols and terminology apply when dealing with actions:

(a) Permanent actions (G). These are the actions that remain monotonic and will vary by a negligible amount with time, e.g. self-weight, fixed equipment, fixed partitions, finishes and indirect actions caused by shrinkage and/or settlement.

(b) Variable actions (Q). These are the actions that do not remain monotonic and may vary with time, e.g. imposed loading, wind, snow and thermal loading.

(c) Accidental actions (A). For example, explosion or impact loading.

Actions are defined by ‘representative’ values and the main representative value used in design is the characteristic value. The characteristic value should, where possible, be derived from the statistical data associated with the action and, depending on the design condition, it will be a mean value, an upper or lower value or a nominal value (this being used when it cannot be derived from statistical data).

i. Characteristic values

- Permanent actions (Gk)

Where this relates to the self-weight of the material, because the variability of the action is small (i.e., the coefficient of variation of the action during the design working life is less than 0.05–0.1), when dealing with timber or wood-related products, Gk, is generally derived using the mean density of the material.

Characteristic values for self-weight, Gk, are obtained from the standards that give mechanical properties.

- Variable actions (Qk)

The characteristic values of the variable actions referred to in Eurocode EN 1991:

‘Actions on structures’ are given in the relevant parts of that code.

When dealing with climatic actions (e.g. wind, temperature etc.), EC0 states that the characteristic value is based on the probability of 0.02 of its time-varying part being exceeded for a reference period of 1 year. The probability of exceedance (p) and the reference period (r) are linked by the approximate relationship T ∼ = rp, (where T is the return period) and represents the likely time between two successive occurrences when the characteristic value will be exceeded. On this basis the return period will be 50 years, which equates to a probability of 0.64 that the characteristic value will be exceeded during this period. This criterion also applies to imposed loading on the floors of buildings.

It should also be noted that where a building carries more than one floor, a reduction in loading is possible and guidance on this is given in EC1.

- Accidental actions (Ad)

Because of the lack of statistical data relating to this condition, the design value, Ad, should be specified and agreed for each project.

These are as follows:

The combination value ($\psi_0 Q_k$): used for verification of ULS and for the characteristic combinations of irreversible SLS.

The frequent value ($\psi_1 Q_k$): used for verification of ULS involving accidental actions and for the verification of reversible SLS.

The quasi-permanent value ($\psi_2 Q_k$): used for the assessment of long-term effects, for the representation of variable actions in accidental (and seismic) combinations at the ULS, and for the verification of frequent and long-term effects of SLS.

The factors ψ_0, ψ_1 and ψ_2 are reduction factors. Factor ψ_0 takes into account the reduced probability of simultaneous occurrence of the most unfavorable values of several independent variable actions. Factor ψ_1 is a time-related function and sets an upper limit for the value of the variable action to which it applies. For buildings, it is set such that the proportion of time it is exceeded is 1% of the reference period.

Factor ψ_2 is also a time-related function and in timber engineering its primary role is to convert variable actions to equivalent permanent actions (referred to as quasi-permanent actions) in order to derive the creep loading on the structure. For floor loading on buildings, the value of ψ_2 is set such that the proportion of time it is exceeded is 50% of the reference period and in the case of wind loading and temperature loading (as well as snow loading for sites at an altitude up to 1000 m above sea level) the value will be zero.

Values for ψ_0, ψ_1 and ψ_2 of the values relevant to loading conditions associated with timber buildings are given in Table 3.1

Table 3.1 Values for ψ factors*

Variable action	ψ_0	ψ_1	ψ_2
Category of imposed loads on buildings (see EC1)			
Category A: domestic and residential areas	0.7	0.5	0.3
Category B: office areas	0.7	0.5	0.3
Category C: areas where people congregate	0.7	0.7	0.6
Category H: roofs (noting that imposed load should not be applied with snow load or wind action (see clause *3.3.2(1)* in EC0)	0.7	0	0
Snow loads on buildings			
for sites with an altitude no greater than 1000 m above sea level	0.5	0.2	0
Wind loads on buildings	0.5	0.2	0

* Based on *Table NA.A1.1*, UKNA to EC0 [4].

3.2.1. Ultimate Limit State Design with CLT

Generally in cross laminated timbers, the cross section consists of at least three planes glued together which are orthogonal in direction of the grains; this is done to increase the strength of bending in both plane directions. Although the orthogonally of the glued boards to each other provides more strength for CLT, but it causes the rolling shear for the boards.

Stress distribution due to bending in CLT and rolling shear of the boards and their formula (applied in European Code) are explained in the next two sections.

3.2.2. Stress Distribution in CLT Cross Section

According to Gerhard Schickhofer's paper (a Software Tool for Designing CLT Elements: 1D-Plate-Design), CLT shows a main load carrying direction.

In European code, Timoshenko beam theory is applied to calculate the bending and shear of CLT cross sections. Therefore the bending stress distribution over the cross section remains also linear as illustrated in figure 3.1.

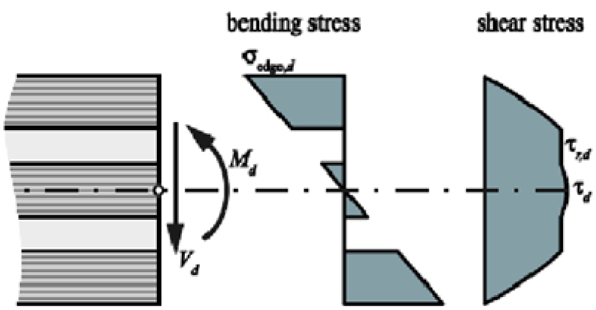


Figure 3.1 Approximated Stress Distribution over Cross Section of a CLT Plate

For verification of bending the condition of $\sigma / f_{m,clt,d} < 1.0$ should be satisfied. In which σ is the maximum stress at the surface of the CLT, and f is characteristic bending strength of CLT.

Since in some countries wood industry no manufacturer produces CLT, the characteristic bending strengths, and modification factor due to load and moisture for this type of material do not yet exist. Due to this gap, the calculation for bending stress of over CLT surface cannot be compared to the strength characteristic value.

3.3. Shear in CLT

When a slab is loaded laterally and subjected to bending, shear stresses will also arise.

In accordance with elastic bending theory, shear stresses will be generated parallel to the longitudinal axis of the slab section and, to achieve equilibrium, equal value shear stresses will be generated in the beam perpendicular to the longitudinal axis as shown in Figure 3.2a.

The value of the shear stress at any level in the cross-section of a slab, as derived from elastic theory, is $\tau = \frac{VS}{Ib}$……………..… (3.1)

Where τ is the shear stress at the required level, V is the shear force at the position being considered, S is the first moment of the area above the shear stress level about the neutral axis, I is the second moment of area of the cross-section about the neutral axis, b is the width of the cross-section at the shear stress level.

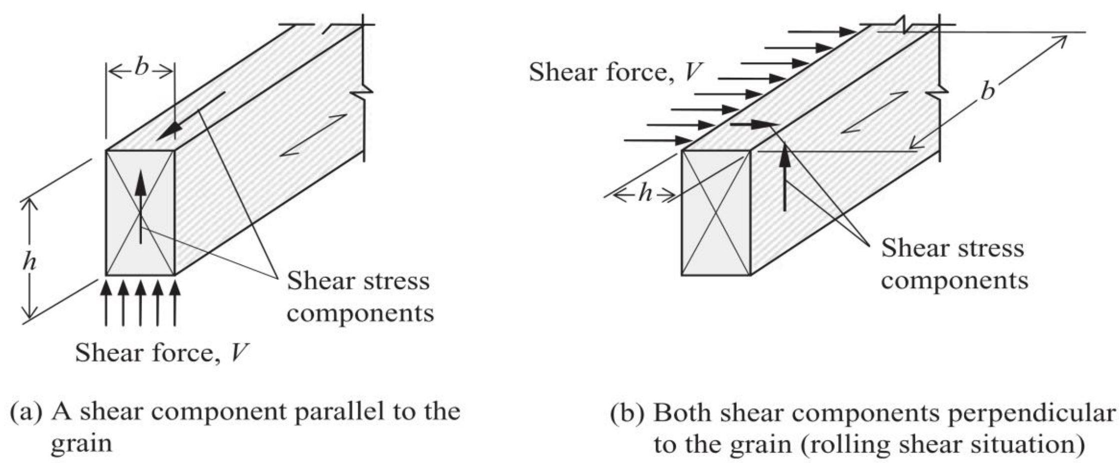


Figure 3.2 Shear stress components in a member: (a) a shear component parallel to the grain; (b) both components perpendicular to the grain (rolling shear).

At any position along the beam the shear stress at the top and bottom faces of the cross-section will be zero and the maximum shear stress will arise at the neutral axis position. For a rectangular section of width b and depth h the maximum shear stress will occur at mid-depth and will be:

$$\boldsymbol{\tau = \frac{3}{2}\frac{V}{bh}} \text{……………..… } \textbf{(3.2)}$$

The limit state for shear design is the ULS and the requirement in EC5 is that for shear with a stress component parallel to the grain or where both shear stress components are perpendicular to the grain (e.g. rolling shear), as shown in Figure 3.2b,

$$\tau_d \leq f_{v,d} \text{ ………………..… (3.2)}$$

Where $f_{v,d}$ is the design shear strength for the condition being investigated, i.e. Figures 3.2a or 3.2b.

When deriving the shear strength of members subjected to bending, the assumption is made in EC5 that the influence of c**racks** in members is ignored.

In the above cases,

V_d is the design shear force acting on the beam. In determining the design shear force, the contribution made by any loading acting on the top face of the slab without a notch and within a distance of **h** from the edge of the support may be ignored due to the effect of the bearing. This also applies

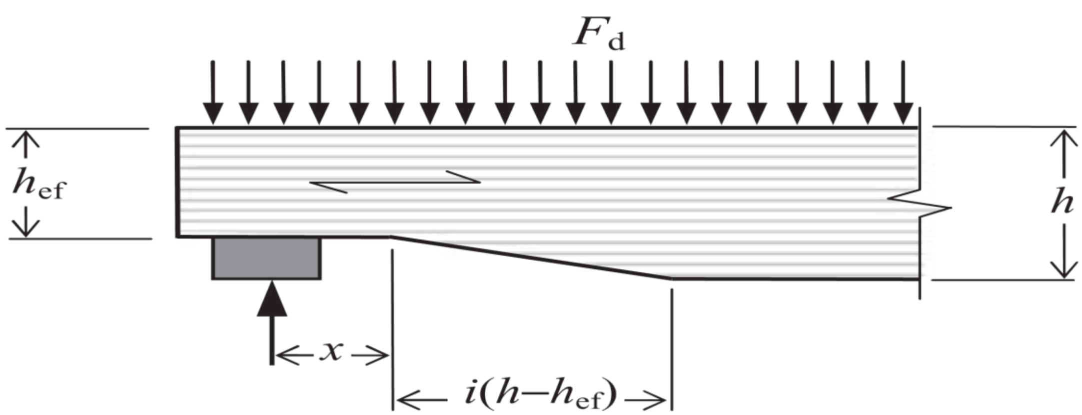


Figure 3.3 Beams with an end notch.

to slab with a notch on the opposite side to the support where the contribution within a distance of h_{ef} from the edge may be ignored. No reduction is permitted when the notch is on the same side as the support (Figure 3.3). $f_{v,d}$ is the design shear strength and is defined as:

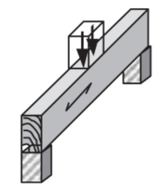
$$\boldsymbol{f_{v,d} = \frac{K_{mod}K_{sys}f_{v.k}}{\gamma_M}} \text{………………………...……………..………..… } \mathbf{(3.3)}$$

Where k_{mod}, k_{sys}, γ_M are as described in Section 3.4.1 and $f_{v,k}$ is the characteristic shear strength. The strength is based on the shear strength parallel to the grain (as this is smaller than the shear strength perpendicular to the g\rain) and for softwood has in general been derived from the bending strength of the species, being the minimum of 3.8 N/mm^2 or $0.2(f_{m,k})^{0.8}$,

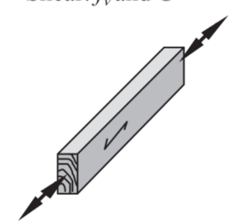
Where $f_{m,k}$ is the characteristic bending strength of the timber. Values for the shear strength of timber and wood-based structural products are given by: table 3.2

Table 3.2 Strength and stiffness properties and density values for structural timber strength classes, (in accordance with Table 1, of BS EN 338: 2003

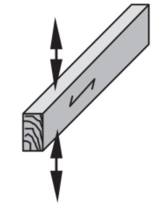
Strength class		Characteristic strength properties (N/mm^2)						Stiffness properties (kN/mm^2)				Density (kg/m^3)	
		Bending ($f_{m,k}$)	Tension 0 ($f_{t,0,k}$)	Tension 90 ($f_{t,90,k}$)	Compression 0 ($f_{c,0,k}$)	Compression 90 ($f_{c,90,k}$)	Shear ($f_{v,k}$)	Mean modulus of elasticity 0 ($E_{0,mean}$)	5% modulus of elasticity 0 ($E_{0.05}$)	Mean modulus of elasticity 90 ($E_{90,mean}$)	Mean shear modulus (G_{mean})	Density (ρ_k)	Mean density (ρ_{mean})
Softwood and poplar species	C14	14	8	0.4	16	2.0	1.7	7.0	4.7	0.23	0.44	290	350
	C16	16	10	0.5	17	2.2	1.8	8.0	5.4	0.27	0.50	310	370
	C18	18	11	0.5	18	2.2	2.0	9.0	6.0	0.30	0.56	320	380
	C20	20	12	0.5	19	2.3	2.2	9.5	6.4	0.32	0.59	330	390
	C22	22	13	0.5	20	2.4	2.4	10.0	6.7	0.33	0.63	340	410
	C24	24	14	0.5	21	2.5	2.5	11.0	7.4	0.37	0.69	350	420
	C27	27	16	0.6	22	2.6	2.8	11.5	7.7	0.38	0.72	370	450
	C30	30	18	0.6	23	2.7	3.0	12.0	8.0	0.40	0.75	380	460
	C35	35	21	0.6	25	2.8	3.4	13.0	8.7	0.43	0.81	400	480
	C40	40	24	0.6	26	2.9	3.8	14.0	9.4	0.47	0.88	420	500
	C45	45	27	0.6	27	3.1	3.8	15.0	10.0	0.50	0.94	440	520
	C50	50	30	0.6	29	3.2	3.8	16.0	10.7	0.53	1.00	460	550
Hardwood species	D30	30	18	0.6	23	8.0	3.0	10.0	8.0	0.64	0.60	530	640
	D35	35	21	0.6	25	8.4	3.4	10.0	8.7	0.69	0.65	560	670
	D40	40	24	0.6	26	8.8	3.8	11.0	9.4	0.75	0.70	590	700
	D50	50	30	0.6	29	9.7	4.6	14.0	11.8	0.93	0.88	650	780
	D60	60	36	0.6	32	10.5	5.3	17.0	14.3	1.13	1.06	700	840
	D70	70	42	0.6	34	13.5	6.0	20.0	16.8	1.33	1.25	900	1080



Bending parallel to grain: f_m and E_0
Shear: f_v and G



Tension or compression parallel to grain: $f_{t,0}$, $f_{c,0}$ and E_0



Tension or compression perpendicular to grain $f_{t,90}$, $f_{c,90}$ and E_{90}

Subscripts used are:0, direction parallel to grain; 90, direction perpendicular to grain; m, bending; t, tension; c, compression; v, shear; k, characteristic.

k_v is a factor that takes account of the effect of a notch in the slab section. Stress concentrations are generated by a notch, and from the application of linear elastic fracture mechanics combined with experimental testing, values for the factor have been derived to remove the risk of beam failure due to the effect of crack propagation. For beams with a notch, EC5, 6.5.2, requires the following: where the notch is on the opposite side to the support , $k_v = 1.0$; where the notch on the same side as the support , k_v is the lesser of:

For shear design of CLT, there are two conditions to be satisfied. One is the classical shear calculation which is $\tau / f_{v,clt,d} < 1.0$, in which τ is the maximum shear stress of CLT. The maximum shear stress of CLT occurs in the center line of the middle layer, as illustrated in Figure 3.1.

The other shear condition that has to be satisfied for design with CLT is $\tau_r / f_{r,clt,d} < 1.0$.

This condition takes care of the rolling shear in the CLT cross section. Rolling shear generally occurs in solid wood panels with cross layers. In CLT every other layers are oriented parallel to each other, when the middle layers are perpendicular in direction. The primary direction of load-bearing capacity for CLT is generally corresponds to the outer layer orientation. When the load is applied to CLT, deformation increases as the applied load is in plane with perpendicular to grain layers (shown in Figure 3.4).

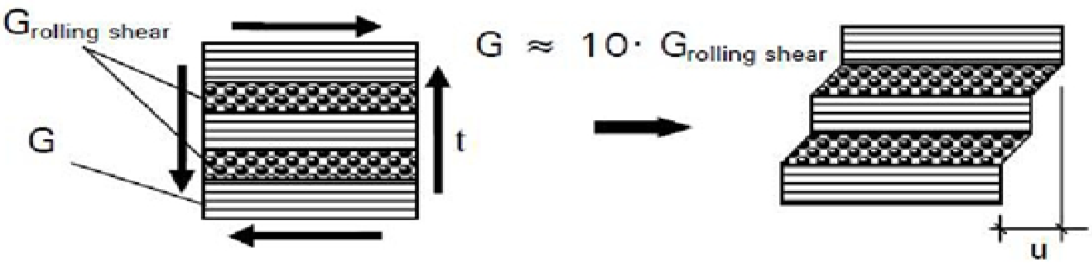


Figure 3.4 Rolling Shear in CLT

G, the rolling shear modulus is one tenth of the regular shear modulus. The distribution of shear the perpendicular oriented layer is shown in figure 3.5

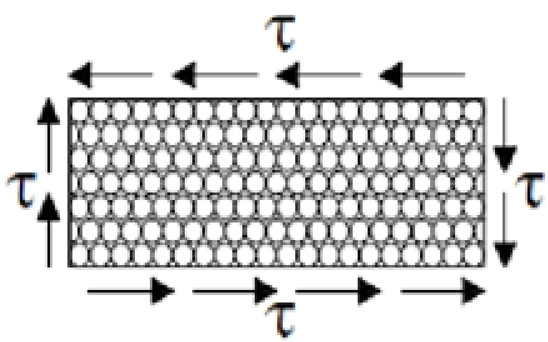


Figure 3.5 Shear Stress Distribution due to Rolling Shear

For the design of CLT the composite theory may be used. The calculation method is based on the strength and stiffness properties of the single layers. Thus, both layers loaded parallel to the grain and cross layers loaded perpendicular to the grain are taken into account.
It must be noted that for design of CLT in both shear condition, characteristic values for shear strengths, $f_{v,clt,d}$ and $f_{r,clt,d}$ are required to be used from Code.

3.3.1. Implementation OF Calculations

At this stage of knowledge about the behavior of CLT when subjected to bending and shear stresses, simplified strength checks can be computed in an Excel spreadsheet using basic laminate beam theory and checking the resulting stresses versus maximum stress failure criteria.
For bending moment resistance calculations, the figure below shows a schematic representation of the elastic modulus, stress and strain distributions in a hypothetical layered non-homogeneous beam.

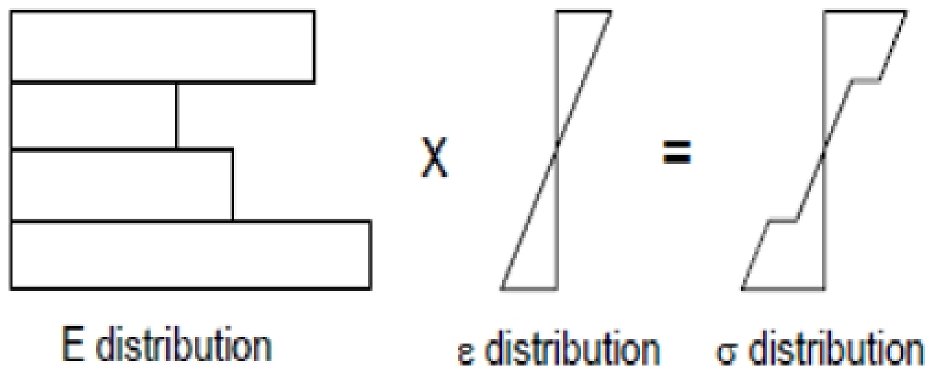


Figure 3.6 Distribution of stress, strain and elastic modulus in a laminated beam

The strain compatibility assumption demonstrated in the figure above requires that there is perfect bonding between the layers and that rolling shear in the perpendicular-oriented wood layers does not result in excessive deformations in that layer. Also, behavior is assumed to lie within the linear elastic range of the material response, which is acceptable for a wood product as they are generally designed to remain within this range.
Once these assumptions are accepted, the stiffness of the CLT section can be calculated by summing the individual stiffness of each lamina (i.e. the in-plane or out-of-plane elastic modulus of that layer multiplied by the moment of inertia of that layer about the mid plane). Maximum strain capacity can be computed using the elastic modulus and the maximum stress capacity of the layer in question. It is then a matter of converting that maximum strain to an

equivalent curvature of the member and computing the moment

Resistance of the section that corresponds to the calculated maximum curvature. See figure below for calculations showing these formulas:

Shear stress capacity can be checked in a more direct manner, using CSA S6-06 code equations for the shear capacity of individual timber members and multiplying that capacity by the number of layers in the direction of loading.

As per CSA S6-06, Cl.9.7.3, the shear force can be computed as shown in the following

Using this approach to computing bending moment and shear force resistance in CLT panels a basic understanding of the capacity of a CLT bridge deck can be obtained.

3.4. Slab Sizing

Preliminary slab sizing was performed using load/span charts from slab manufacturers. Other issues also affect the dimensions of the slab, and the preliminary slab choices are hence verified against the following requirements: bearing failure, shear failure, robustness, and fire-resistance.

3.4.1. Bearing failure

Bearing failure is the localized compression failure of a spanning element or its supports at the area of interface. Designing against this ensures that there is sufficient area over which the element is supported so that the stress there does not exceed the maximum allowable bearing stress.

- The symbols used in EC5 for compressive strength properties parallel and perpendicular to the grain are as follows:
  - $\boldsymbol{\sigma_{c,0,k}}$: characteristic compressive strength parallel to the grain.
  - $\boldsymbol{\sigma_{c,90,k}}$: characteristic compressive strength perpendicular to the grain.
- The subscripts:
  - c (or t) refers to the type of stress (c – for compressive; t – for tensile).
  - 0 (or 90) refers to the direction of the applied stress relative to the grain direction
  - (0 – parallel to the grain direction; 90 – perpendicular to the grain direction).
  - K (or d) refers to the nature of the stress (k – characteristic strength; d – design strength).

When timber is compressed perpendicular to the grain the wood fibers, 59 which can in principle be likened to a bundle of narrow thin-walled tubes loaded laterally, with stand increased loading as they are squeezed together and as they start to collapse the rate of load increase reduces. This behavior continues until the fibers are fully squashed and as the wood is strained beyond this stage the sustained load will continue to rise until eventually failure occurs, usually by shearing across the grain. The strain in the wood can exceed 30% and failure may still not arise. To control deformations at the failure condition, BS EN 1193 requires the compression strength of timber perpendicular to the grain to be determined using a 1% strain offset. However, where the bearing width extends over the full width of the member, depending on the position of the bearing area, the bearing length and the member depth, this strain limit can be exceeded and the rules in EC5 allow the bearing strength to increase by up to a factor of 4. At this value, the strain will be of the order of 10% and the effect of the associated increase in deformation at the bearing on the structure must be taken into account in the design. Examples of bearing effects (failures) are illustrated in Figure 3.7. For compression perpendicular to the grain the condition to be met for design compressive stress at supports ($\sigma_{c,\ 90,d}$) is [2]

$$\sigma_{c,90,d} \leq k_{c,90}\, f_{c,90,d} \quad \text{.................................. (3.4)}$$

Here $\boldsymbol{\sigma_{c,90,d}}$ is the design compressive stress perpendicular to the grain = $\mathbf{V_d/bl}$, where $\mathbf{V_d}$ is the design bearing force, **b** is the bearing width, **l** is the bearing length and $\mathbf{k_{c,90}}$ is a factor depending on the relative positions of the supported and supporting members and also the type of support - this assumes an allowance of increased strain at failure, but can be conservatively taken as **1.0** . $\mathbf{f_{c,90,d}}$ is the design compressive strength perpendicular to the grain and is defined as:

$$f_{c,90,d} = \frac{K_{mod}.\mathrm{K}_{sys}.f_{c,90,k}}{\gamma_M} \quad \text{....................................... (3.5)}$$

Where

$f_{c,90,k}$ characteristic compressive strength (perpendicular to grain)

k_{mod} modification for load duration and moisture content

k_{sys} system strength factor

γ_M Material partial safety factor (= 1.25 for Glulam/CLT)

Contact extends over the full member width, and the support conditions are in line with the requirements given in EC5, the value can be increased up to a maximum of 4.0, but, as previously stated, at this value the compressive deformation will be approximately 10% of the member depth and must be taken into account in the design. For the case where a beam of breadth b and depth h rests on isolated supports or where a member is supported continuously, the factor is calculated as described in the following sections.

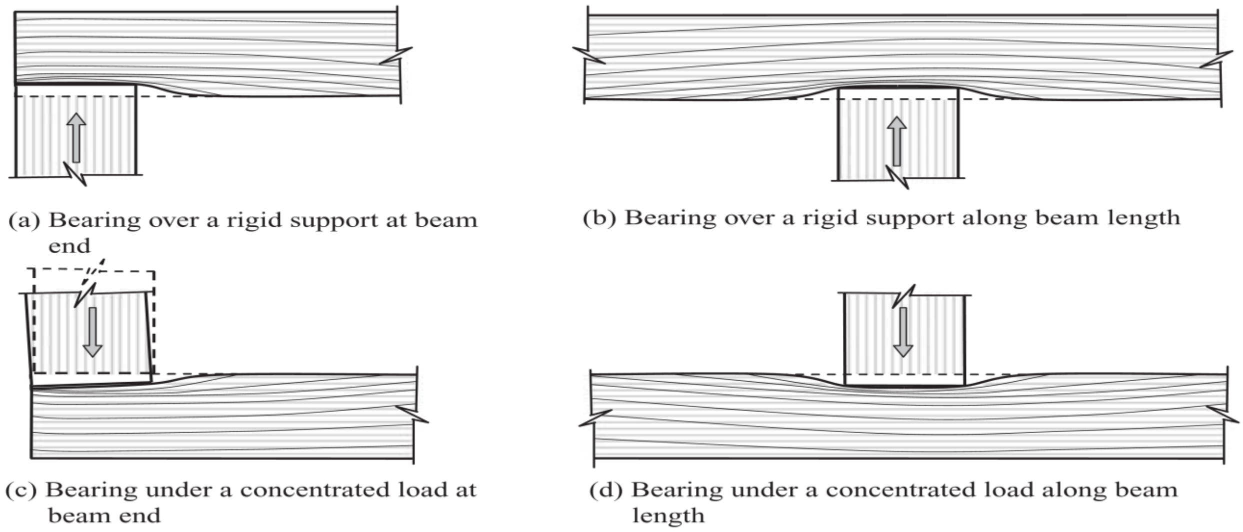


Figure 3.7 Verification of Slab Suitability in Fire

In order to assess the behavior of the floor slabs and calculate they have the required fire resistance, the method from Structural Design for Fire Safety is utilized. This incorporates the Eurocode method of reduced properties for the load-bearing assessment, a check on the time to integrity failure, and the empirical formula for time to failure derived by Janssens for solid wood decks. Janssens' method may be the most appropriate since it assumes a model of a solid deck consisting of multiple thinner layers - exactly as is the case in reality for cross-laminated timber. The insulation criterion need not be checked if the stability and integrity criteria are met because there will be sufficient floor section left to prevent excessive temperature rises [3].

3.4.2. Loading in Fires

Loading on the structure during a fire is likely to be less than that under normal operation - the permanent load and some of the imposed load. Design codes take account of this fact by reducing the load factors so that for the Eurocode, the design load in fire, $F_{d,f}$, is

$$F_{d,f} = G_k + 0.5Q_k \quad (3.6)$$

Where G_k is the permanent load and Qk is the imposed load (3)

3.4.2.1. Janssens' method

The empirical formula for the time to structural failure, t_{sf} (minutes), derived by Janssens for design purposes is given by

$$t_{sf} = 1.25d\left(1 - \sqrt{0.4R_A}\right) - 11.30 \quad (3.7)$$

Where **d** is the slab thickness (mm), and $\mathbf{R_A}$ is the ratio of applied load (in normal conditions) to design strength. This will assess the load bearing criterion, and is based on transformed section. Here, the applied load used is that of the normal situation.

3.4.2.2. Eurocode Method

The Eurocode Method is a time dependent verification of the element suitability. It assumes the charred section of a structural element has no load bearing capacity, and as such only the un charred section contributes. The depth of char is calculated at the time duration specified, and the strength of the remaining section can then be found. The depth of char, c, in one dimensional burning is given by

$$c = \beta t \qquad (3.8)$$

Where β the charring rate (mm/min) and t is is the required resistance time (minutes). With this, the reduced section depth, d_f, can be found and the reduced section properties (Area and Section Modulus) can be calculated.

The design strength of the remaining section, M_f, is given by

$$M_f = K_{mod,fi}.\frac{K_{fi}.f_b.z_f}{\gamma_M,f_i} \qquad (3.9)$$

Where,

$k_{mod,fi}$ = $1.0 - \frac{1}{200}\frac{p}{A_r}$ (for bending - EC5 part 1-2, 4.2.3(3))

P perimeter of cross -section exposed to fire

Ar reduced section area

k_{fi} =1.15 for Glulam (Table 2.1 of EC5 part 1-2)

f_b characteristic bending strength

Z_f reduced section modulus

γ_M,fi partial safety factor for timber in fire (= 1.0, EC5 part 1-2, 2.3(1))

This must be greater than the fire loading for the slab to achieve the fire rating.

To check the integrity of the flooring system, EC5 has a further reduction factor to account for the type of joint detail between floor slabs,ξ, and it is found from Table E6 of EC5 part 1-2. This factor is applied to the time to burn through the whole slab depth under a notional char rate, βn (=0.7mm/min for Glulam - Table 3.1, EC5 part 1-2). Hence the time to integrity failure, t_{if} (minutes), is given by:

$$\boldsymbol{t_{if} = \frac{\xi_d}{\beta_n}} \qquad (3.10)$$

Where **d** is slab depth (mm).

CHAPTER FOUR

4.1. Design of Asymmetric slimflor Beams

4.1.1. Ultimate Limit State Design with Asymmetric slimflor Beams

Owing to the lack of available test data on the performance of ASB sections with Timber slabs, the ultimate design resistance is based on the bare steel ASB section alone. However, the beams may be designed compositely for the serviceability condition, provided that **Fasteners** the appropriate **coach bolt with nut** is provided (this would normally be needed for robustness requirements).

The design of the beams involves checking the construction stage and at the normal stage, ultimate limit and serviceability limit states. In both these stages, the ASB could be subject to torsion and bending. Unequal CLT Unit spans may also cause significant torsion.

Design loading cases are presented in detail in the following sections for internal beams. Loading cases for edge beams, or beams where the CLT Unit span direction changes i.e. from perpendicular to the ASB on one side to parallel on the other, will be variations on the cases presented, and are likely to be dominated by torsion.

Worked examples are provided in Appendix A.

4.1.1.1. Construction stage

The ASB sections are designed according to whether they are used in construction (without superimposed load). Three cases are considered for the internal beams as shown in Figure 4. Where the CLT Unit spans differ, the longer side is assumed to be on the left hand side in the figures. It should be noted that it is not necessary to increase the torsion by using different partial safety factors for dead loading each side of the ASB, as the variation will be very modest.

I. Loading cases for Type 1 construction

The sequence of erection for construction is likely to lead to the following design cases, as shown in Figure 4.1:

i. CLT Units and construction loading on one side only.
ii. CLT Units on both sides, construction loading on one side.
iii. CLT Units and construction loading on both sides.

Case (ii) is unlikely to be critical unless the CLT Unit spans are unequal.

The principal design checks are for:

Cases (i) & (ii) lateral torsional buckling (LTB) ~ bending and torsion

Local capacity ~ bending and torsion

Twist ~ horizontal displacement of top flange (SLS check)

Case (iii) buckling (LTB) ~ bending

(Include torsion and carry out local capacity check if PC unit spans are unequal.)

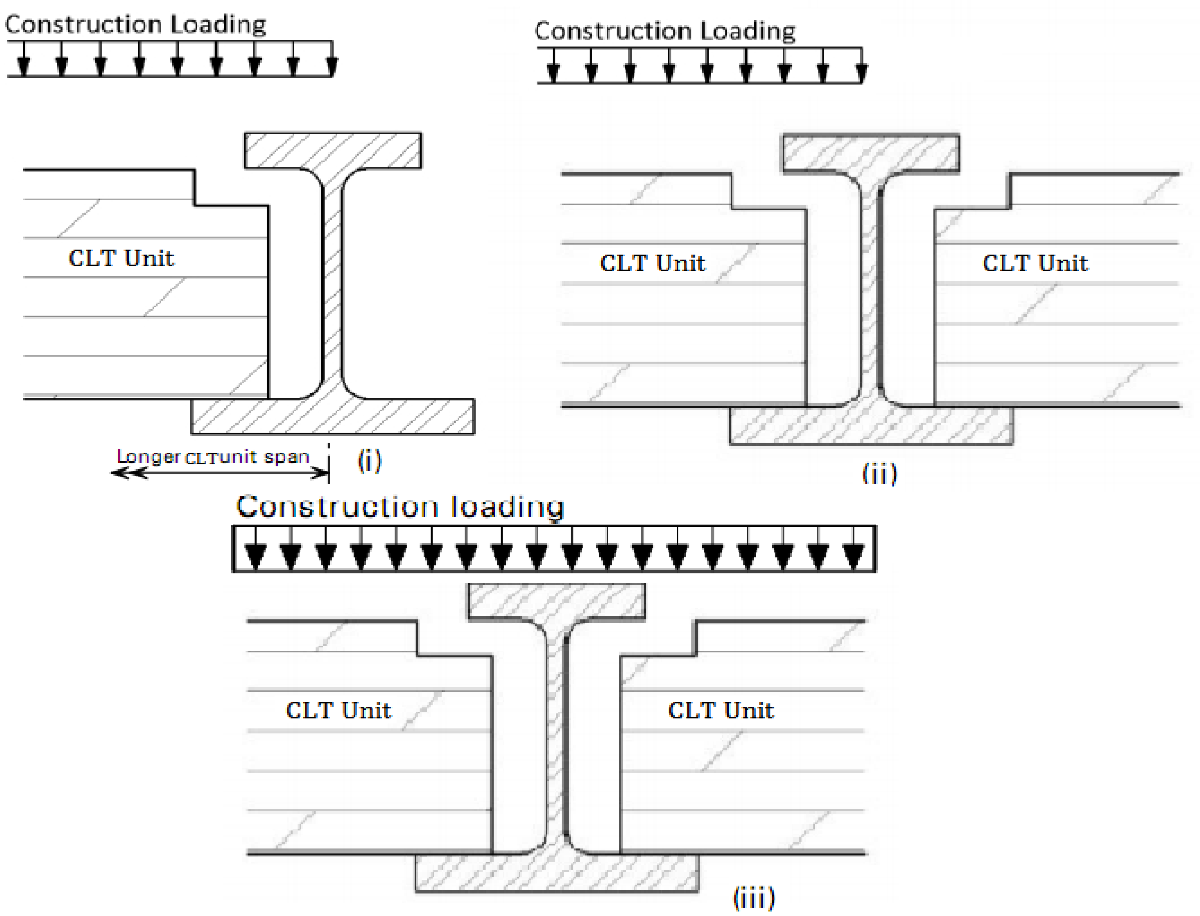


Figure 4.1 Design load cases for the construction stage

4.1.1.2. Normal Stage

As with the construction stage, ASBs are designed according to whether they are used in construction. Two cases are considered for Normal Stage: patterned imposed load and uniform imposed loading. Full lateral and torsional restraint is possible in construction, provided a certain depth of encasement is achieved (see Section 4.4).

I. Loading cases for Type 1 construction

The loading cases for design at the normal stage for Type 1 construction are:

i. CLT Units and super dead load on both sides and imposed load on one side

ii. CLT Units and super dead load and imposed load on both side

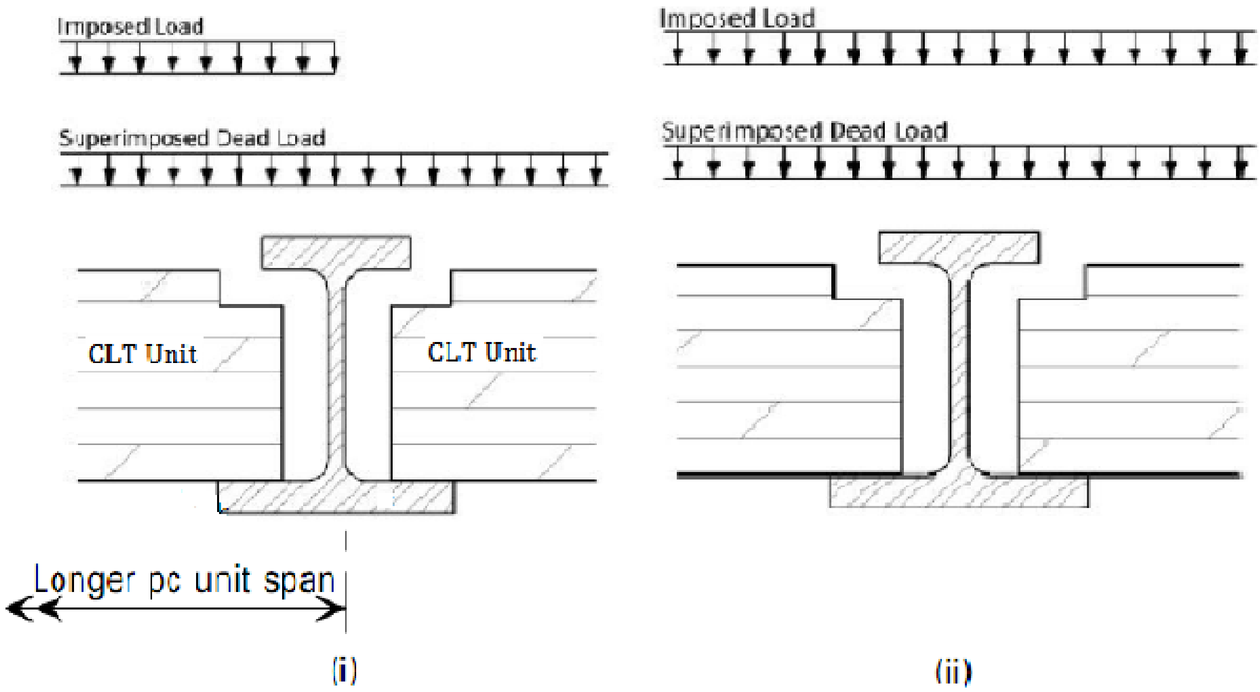


Figure 4.2 Normal stage design load cases for construction

The principal design checks are for:

Case (i) buckling (LTB) ~ bending and torsion (if restraint is assumed, this check does not apply - see Section 4.4)

Local capacity ~ bending and torsion

Case (ii) buckling (LTB) ~ bending (if restraint is assumed, this check does not apply- see Section 4.4)

- ❖ Shear capacity check ~ ASB
  ~ CLT Unit
- ❖ Bending capacity
- ❖ Fire resistance
- ❖ Dynamic response (at SLS)
- ❖ Deflections (at SLS)
- ❖ Irreversible deformation (stress check) (at SLS)

(Include torsion in the buckling check, and carry out a local capacity check, if the CLT Unit spans are unequal. When this applies, the bending capacity check may be omitted because it is a plastic check and will always be less onerous than the local capacity check, which is based on elastic properties.)

4.1.2. Requirements for limited composite behavior

Non-composite behavior is assumed for Type 1 construction and edge beams, because of insufficient stiffener encasement. Composite behavior is not assumed to occur when calculating the ultimate resistance in the normal condition, owing to the lack of test data on this form of construction.

Given the conservative assumptions made above, it is considered reasonable to ignore the presence of the cores in the calculation of the composite stiffness when the PC Units are perpendicular to the ASB. When the CLT Units are parallel to the ASB, or they are parallel on one side (and perpendicular on the other), it is recommended that non-composite behavior is assumed.

4.1.3. Stability of the compression flange

When CLT Units are placed on the bottom (tension) flange of a beam, no restraint is provided to the compression flange, and so lateral torsional buckling is possible and must be checked. The friction between the CLT Unit and the bottom flange affords partial lateral restraint to the tension flange, but this is ignored in this publication. When the ASB section is fully encased to the top (compression) flange by grouting, it cannot buckle laterally, and so full restraint against buckling can be assumed. Restraint details necessary to achieve this are shown in Figure 4.3

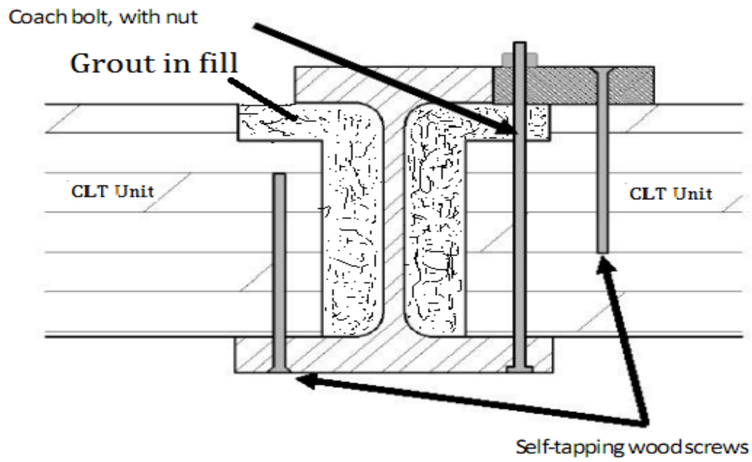


Figure 4.3 Restraint conditions to prevent both lateral torsional buckling and lateral distortional buckling

Even when the section is partially encased up the web with compacted grout significant restraint can occur. This is because twist and lateral movement of the encased part of the section is restricted. The grouted units can act like a stiff diaphragm in the plane of the floor and can provide the necessary reaction forces, provided there is sufficient strength in the floor. However, the protruding web and compression flange could potentially twist and buckle laterally, so that the shape of the section is distorted, as shown in Figure 4.4. This is known as lateral distortional buckling, and the protruding section should be checked for this, or the beam designed as unrestrained.

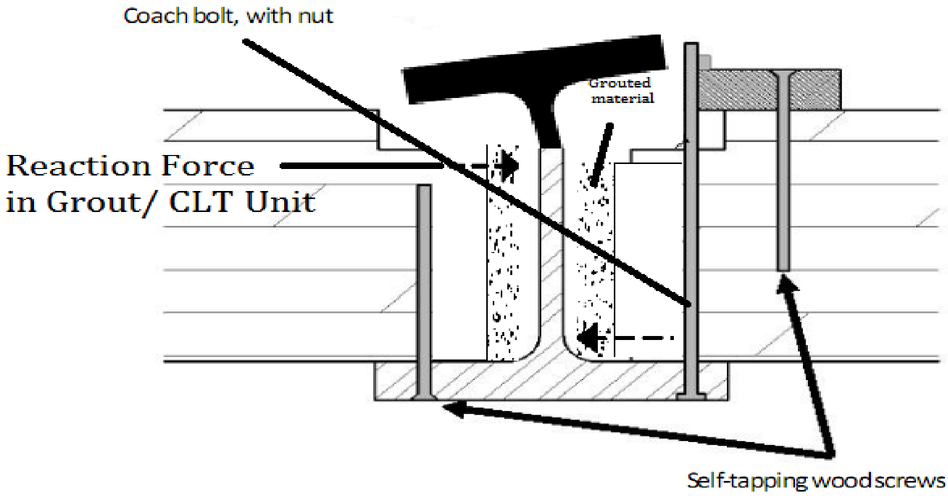


Figure 4.4 Figure 4.4 Lateral distortional buckling of ASB section

For a partially encased condition, the designer has the choice of assuming no restraint, and designing for lateral torsional buckling, or designing for lateral distortional buckling and confirming that the restraint system is adequate.

The susceptibility of a section to lateral distortional buckling may be analyzed by calculating the elastic buckling moment for a given unrestrained length. It may be expressed in terms of an equation incorporating the equivalent slenderness, λ_{LT}, which can be used in the BS5950 rules. The analysis method was developed for symmetric I-sections but has been extended in a study by the SCI to cover asymmetric sections. The methodology is explained below briefly and more fully in Appendix C.

Unlike lateral torsional buckling, where a beam will always have a propensity to form a single half-wave buckle over the full unrestrained length, lateral distortional buckling can involve the formation of several half-wave buckles over the available unrestrained length. The half-wave buckling length for the least energy solution for a given section is known as the elastic critical buckling length, which may be greater, or less, than the unrestrained length. It can be shown that the **elastic critical buckling length** for asymmetric sections subject to lateral distortional buckling, based on a single half-wave buckle, is given by:

$$L_{\mathrm{cr}} = 3.74\left\{2 \times\} I_{yc}\left(\frac{D}{t}\right)^{3}\right\}^{0.25} \quad \ldots \ldots (4.1)$$

Where:

I_{yc}	is the second moment of area of the top flange about its major axis
D	is the depth of the section
t	is the web thickness

At an unrestrained length equal to this, the buckling resistance is equal to the minimum elastic critical buckling moment. For unrestrained lengths less than this, the elastic buckling moment rapidly increases with decreasing length. For unrestrained lengths greater than this, the analysis again shows an increase in elastic buckling moment, but classical buckling theory will show that multiple waves are possible at certain lengths at the same minimum buckling moment. Therefore, to be conservative, a buckling resistance based on an unrestrained length equal to the elastic critical buckling length is assumed for design here.

It can be shown that an important parameter in lateral distortional buckling of I- sections is:

$$\left(\frac{D}{t}\right)^{0.75} \quad \ldots \ldots (4.2)$$

For the current range of ASB sections, it is found that the critical buckling length L_{cr} varies in the range:

$$222 \text{ to } 323 \text{ times } \left(\frac{D}{t}\right)^{0.75} \quad \ldots \ldots (4.3)$$

When expressed in units of mm. This results in critical buckling lengths between 1.5 and 2.7m, which is well within normal beam spans. It is also found that the equivalent slenderness λ_{LT} at the critical buckling length varies in the range

$$3.3 \text{ To } 3.8 \text{ times } \left(\frac{D}{t}\right)^{0.75} \quad \ldots \ldots (4.4)$$

This results in values of λ_{LT} between 20 and 40. The limiting slenderness (λ_{L0}) of grade S355 sections to BS 5950-1 is approximately 30. Therefore, it is seen that for sections where $\lambda_{LT} < 30$ there would be no reduction in the bending resistance from the full plastic resistance, and for those ASB sections where $30 < \lambda_{LT} \leq 40$ there could be a slight reduction (<8%) in the bending resistance. However, since the restraint against lateral distortional buckling for this form of construction involves a significant reduction in the depth of the unrestrained portion of the web, it can be concluded that the likelihood of lateral distortional buckling before the full plastic resistance is attained is negligible for the current range of ASB sections, provided that the restraint from the floor is adequate.

The adequacy of the floor may be checked by ensuring that it can provide the necessary restraint forces. Provided that the depth of the CLT Unit (d_{HC}) is greater than half the depth between the flanges (d_1), the reaction force in the floor at the top of the CLT Unit can be taken as 5% of the force in the compression flange, spread over the beam span (Figure 4.5).
For practical reasons, it is recommended that this restraint is only used in design when the depth of the hollow-core unit is at least 0.5*d*1, (see Figure 4.5). This will always be true for 150-260 mm deep CLT Units.

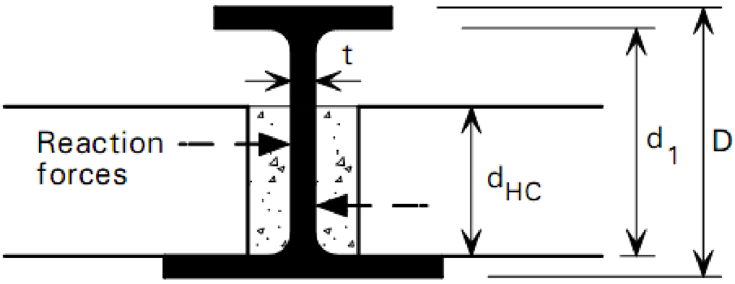


Figure 4.5 Geometric limits to prevent lateral distortional

The ASB section will undergo rotation in the construction condition and (possibly) in the normal stage, owing to pattern loading, cladding or unequal CLT Unit spans. Edge beams can be subjected to considerable twist. An acceptable degree of rotation is a matter for the designer to consider according to the nature of the construction, so that neither the structural behavior nor the finishes are impaired.
In the construction condition, the varying rotation down the length of the beam may be regarded as an imperfection, and a limit on the lateral movement on the top flange of span/500 is recommended. Designers should be satisfied that the lateral deflection is also acceptable in the normal condition (for example, this may be important for hand-rails directly attached to the beam in atria).

4.1.4. Local transverse bending of the bottom flange and web

Transverse bending of the bottom flange will occur because it is acting like two symmetric cantilevers in the transverse direction, as shown in Figure 4.6. This effect will be coexistent with bending in the longitudinal direction and any torsion that is present.

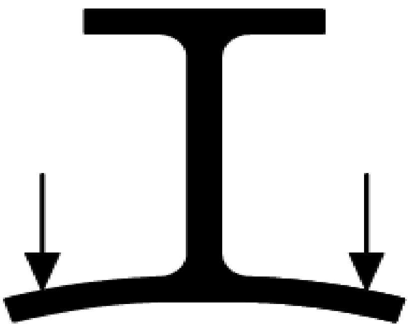


Figure 4.6 Transverse bending of the bottom flange

In slim floor construction, it can conservatively be assumed that the loading is always applied directly through the CLT Units. The point of application of the load is then the center of bearing of the CLT Unit on the bottom flange. The lever arm from this point to the web causes a local bending moment, increasing linearly to its maximum at the center of the web.

For construction, local transverse bending is checked at both the Construction and Normal stages.

An allowance can be made for the effect of this transverse bending on the longitudinal strength of the section by deriving a reduced longitudinal bending strength, but generally this effect is very small (< 3%) for ASB sections.

If it is desired to check the **interaction of transverse bending** with the other effects, one approach is to combine transverse and longitudinal bending stress using Von Mises yield criterion; this is explained in SCI publication P110. An approximate interaction formula is given below.

For the bottom flange

$$\frac{\sigma_1}{P_y} \approx \left[1 - 0.52\left(\frac{M_t}{M_{R,flange}}\right) - 0.48\left(\frac{M_t}{M_{R,flange}}\right)^2\right]^{0.5} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(4.5)}$$

σ_l is the reduced available longitudinal bending strength in the bottom flange

M_t is the applied local bending moment per unit length of the bottom flange caused by an eccentric force at a distance, e, from the web-flange junction

$M_{R,flange}$ is the bending resistance of the bottom flange

(= $1.2 \cdot Z_E = 1.2T\,b^2 p_y/6$ per meter length of beam).

The tensile strength, σ_l, may be used to replace p_y in the bottom flange in a modified plastic analysis of the cross-section or, more usually, used to recalculate a modified section modulus to be used with p_y in strength calculations. A similar approach can be used for the part of the web, by substituting $M_{R.web}$ (= $1.2 \cdot t^2 p_y/6$) for $M_{R.flange}$. These bending resistances, based on 1.2 times the elastic resistance, are slightly less than the plastic bending resistance in both cases; this reduction avoids irreversible deformation under serviceability loads.

When checking the flange interaction, it is sufficient to use a lever arm 50% into the root radius because of the thickening of the section in this area.

4.1.5. Bending and torsion

4.1.5.1. Combined checks

Many of the load cases that need to be checked involve the combined action of bending and torsion. When these actions are assumed to be resisted by the bare steel section alone, the method of analysis set out in publication SCI publication P057 may be followed. It is repeated below for an ASB section.

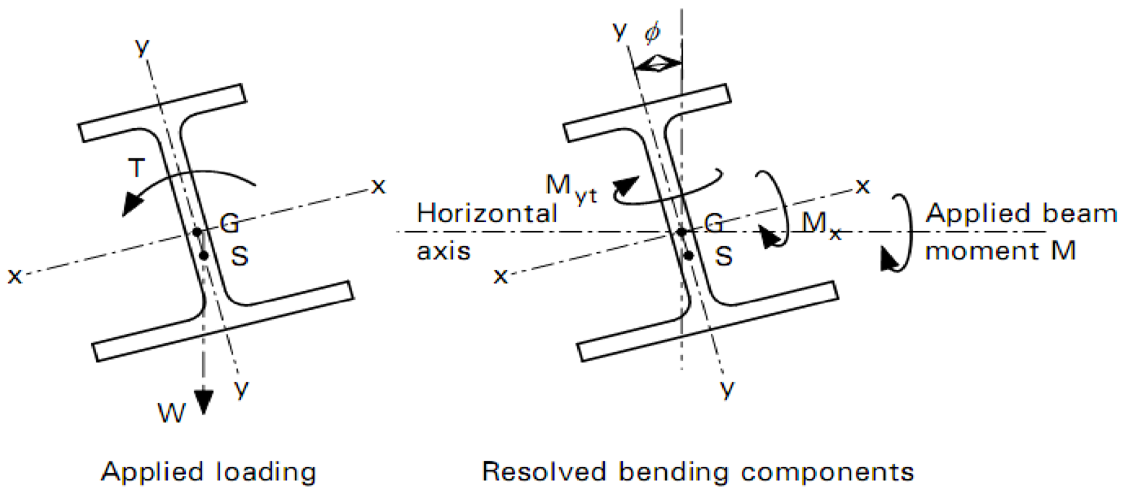


Figure 4.7 Combined bending and torsion applied to the ASB section and resolved bending components induced

The applied loading may be expressed as a torque T about the shear center (S), and a load W acting through the shear center, as shown in Figure 4.7. The loads will cause bending M about the horizontal axis and will induce a twist ϕ in the section. M may be resolved into two orthogonal components about the principal axes of the section, such that:

$Mx = M \cos\phi \approx M$ about the major (x-x) axis of the section and

$Myt = M \sin\phi \approx \phi M$ about the minor (y-y) axis of the section.

These give rise to bending stresses $\boldsymbol{\sigma_{bx}}$ and $\boldsymbol{\sigma_{byT}}$, and warping stresses $\boldsymbol{\sigma w}$, as shown in

Figure 4.8. The y-y bending and warping stresses are greatest at the tips of the flanges and, because of the asymmetry, the most onerous combination of stresses is normally found in the top flange.

The following checks can be made:

- Resistance to buckling.
- Local capacity.
- Resistance to shear stresses from bending, torsion and warping.

The shear check is not normally relevant for ASB sections because the sections are stocky. For further advice on this see P057.

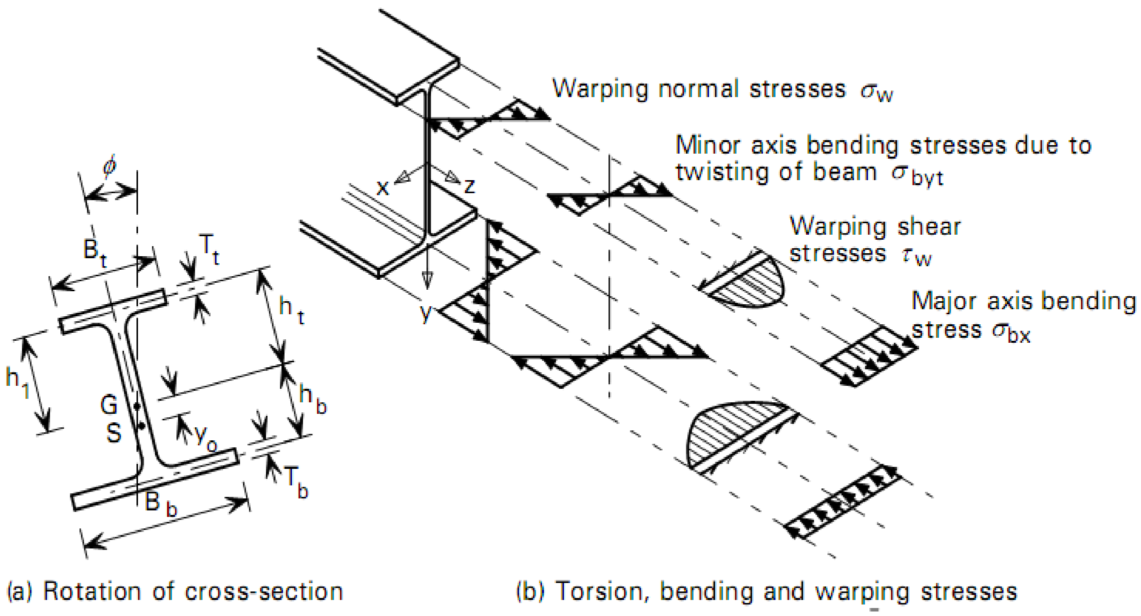


Figure 4.8 Stresses in the ASB section

For the buckling check, the following criterion should be satisfied

$$\frac{M_x m_{LT}}{M_b} + \frac{\sigma_{byt}+\sigma_{byt}}{P_y}\left[1 + 0.5\frac{M_x m_{LT}}{M_b}\right] \leq 1.0 \text{.........................} \quad (4.6)$$

Mx is the maximum applied major axis moment in the segment

m_{LT} is the equivalent uniform moment factor for lateral torsional buckling, given in Clause 4.3.6.6 of BS 5950-1.

[Note: m_{LT} = 0.925 for a non-destabilizing uniformly distributed load, but may be taken conservatively as 1.0]

M_b is the buckling resistance moment of the beam between restraints

[Note: *M*b may be reduced in a more sophisticated analysis where local transverse bending is included, by modifying the section modulus]

σ_{byt} is the bending stress in the flange tips, given by $\sigma_{byt} = M_{yt}/Z_y$

M_{yt} is the resolved minor-axis bending arising from twist

Z_y is the elastic modulus about the minor-axis of the steel section

ϕ is the maximum angle of twist in the beam segment (in radians)

σ_w is the warping stress, given by $\sigma_w = - E\, W_{no}\phi''$

E *is the Young's modulus of elasticity for steel*

W_{no} is the normalized warping function at the compression flange tips, given by $W_{no} = h_1 B_t/2$

Where:

h_1 is the distance from the center of the top flange to the center of gravity

y_o is the distance between the center of gravity and the shear center, given by:

$$y_0 = (h_b I_{yt} + h_t I_{yc})/(I_{yt} + I_{yc}) \ldots\ldots\ldots\ldots \quad (4.7)$$

h_b is the distance from the center of gravity to the center of the bottom flange

I_{yc} is the second moment of area of the top (compression) flange about its major axis, given by:

$$I_{yc} = \frac{T_t B_t^3}{12} \ldots\ldots\ldots\ldots \quad (4.8)$$

I_{yt} is the second moment of area of the bottom (tension) flange about its major axis, given by:

$$I_{yt} = \frac{T_b B_b^3}{12} \ldots\ldots\ldots\ldots \quad (4.9)$$

ϕ'' is the second derivative of ϕ with respect to the distance from a support

p_y is the yield stress of the steel.

The local capacity of the cross-section should be checked. The following criterion is given in P057: $\boldsymbol{\sigma_{bx} + \sigma_{byt} + \sigma_w \leq p_y}$ ………… *(4.10)*

Where: σ_{bx} is the major-axis bending stress, given by $\sigma_{bx} = M_x / Z_x$

Z_x is the elastic modulus about the major-axis of the steel section.

Note: this local capacity check uses a model based on the simple elastic addition of stresses, whereas the major axis (pure) bending check of $M_x < M_{cx}$ (see Section 4.7.1) is a plastic check. This leads to the anomaly that when a section is subject to very slight torsion it apparently has a much reduced bending resistance.

4.1.5.2. Angle of twist

The angle of twist ϕ and its derivatives may be evaluated using the methodology given below, which is based on P057. The variation of twist along the length of a beam depends on the magnitude of the torsion, the boundary conditions and the torsion properties of the section. The usual boundary conditions for beams with bolted end connections are 'fixed' for twist and 'free' for warping. The twist can then be described in the form of equations as a function of the torsion bending constant 'a' and the length 'z' along the beam.

The torsional bending constant is given by:

$$a = \left(\frac{EH}{GJ}\right)^{0.50} \quad \ldots\ldots\ldots\ldots \; (4.11)$$

Where: *G* is the shear modulus of elasticity for steel

H is the warping constant.

J is the torsional constant.

The angle of twist and its derivatives may be found from Table 4.1, according to the ratio of the span to the torsional constant *L/a*.

Table 4.1 Torsional functions for beams with a uniform torque and ends fixed against torsion, free to warp

L/a	$\dfrac{\phi GJ}{T_q a}$	$-\dfrac{\phi'' GJa}{T_q}$
0.0	0.000	0.000
0.5	0.002	0.061
1.0	0.012	0.113
1.5	0.036	0.152
2.0	0.074	0.176
2.5	0.124	0.188
3.0	0.183	0.192
3.5	0.248	0.189
4.0	0.316	0.184
4.5	0.387	0.176
5.0	0.458	0.167
5.5	0.529	0.159
6.0	0.600	0.150
8.0	0.880	0.120
10	1.151	0.099
12	1.417	0.083
14	1.679	0.071
16	1.938	0.062
18	2.194	0.056
20	2.450	0.050
22	2.705	0.045
24	2.958	0.042
26	3.212	0.038

For intermediate values of L/a, linear interpolation is permitted.

4.1.5.3. Buckling resistance

The calculation of the buckling resistance moment, *M*b, is set out in BS 5950-1. *M*b is a function of the slenderness (λ_{LT}) of a beam for lateral torsional buckling, which is given by:

$\lambda_{LT} = uv\lambda\sqrt{(\beta_w)}$ ……….. *(4.12)* In which $\lambda = \frac{L_E}{r_y}$ ……….. *(4.13)*

Where:

L_E is the effective length for lateral torsional bucking

r_y is the radius of gyration about the minor axis

u is the buckling parameter

β_w is a ratio that depends of the section classification and may be taken as:

= 1.0 for Class 1 and 2 sections (which applies to the current ASB range of sections).

The slenderness factor, v, may be determined from Table 19 of BS 5950-1 as function of λ/x and η, or is given for mono-symmetric I-sections by:

$$v = [\left(4\eta(1-\eta) + 0.05\left(\frac{\lambda}{x}\right)^2 + \psi^2\right)^{0.5} + \psi]^{-0.5} \quad \text{……………. } (4.14)$$

where:

η $= I_{yc} / (I_{yc} + I_{yt})$

ψ is the *monosymmetry* index

(I_{yc} and I_{yt} are *defined* earlier)

x is the *torsional* index.

The effective slenderness λ_{LT} is used to determine the bending strength of the beam, p_b, in Table 16 of BS 5950-1[4].

The buckling resistance moment is given by:

Class 1 or 2 sections

$$M_b = p_b S_x \quad \text{………………………. } (4.15)$$

Where:

S_x is the plastic modulus of the section (reduced for local transverse bending where considered necessary).

M_b may then be input into the combined buckling equation, or, where there is no torsion, used to check $M_x \le M_b/m_{LT}$

4.1.5.4. Resistance check for bending only

When there is no torsion, the maximum applied moment, M_x, is compared to the moment capacity, M_{cx}, in accordance with BS 5950-1, such that: $M_x \le M_{cx}$

The moment capacity is reduced when the applied moment is coincident with high shear (applied shear > 60% shear resistance), but for simply supported beams subject to a UDL this is not normally the case. The webs of the ASB sections are also stocky, so shear is unlikely to be a problem.

Currently, all the ASB sections are class 1 plastic, and so the capacity is given by:

$$M_{cx} = p_y S_x \ldots\ldots (4.16)$$

(Where S_x may be reduced for transverse bending)

4.1.6. Serviceability conditions

There are six design criteria at the serviceability limit state:

- A limit on the horizontal movement of the top flange during erection
- A limit on deflection due to imposed load.
- A limit on the total deflection.
- A limit on the twist under patterned imposed loading.
- Avoidance of irreversible deformation.
- Avoidance of excessive vibrations.

Elastic section properties should be used in all serviceability calculations. As well as calculating the vertical deflection of the beam under un factored loading, the angle of twist of the beam under patterned loading should also be calculated (see Section 4.1.5).

4.1.6.1. Elastic properties of composite section

The second moment of area of the composite section is established by transforming the cross-sectional area of Timber into an equivalent area of steel, by dividing by the modular ratio, n.

The modular ratio, n, is a ratio of the Young's Moduli of the two materials in the composite section and is used to make their different properties compatible in calculations. By using the modular ratio it is possible to transform the cross-section area of one material into an equivalent area of the other material. This additional equivalent area is used to calculate the enhancement in behavior of the two materials acting compositely versus the one material acting alone.

To transform the timber slab into an equivalent area of steel, the modular ratio is given by,

$$n = \frac{E_{steel}}{E_{timber}} \quad , \quad \ldots\ldots (4.17)$$

Where: n is a ratio of the Young's Moduli of the two materials in the composite section

E_{steel} is Steel Modulus of elasticity

E_{Timber} is Timber Modulus of elasticity

Due to its susceptibility to creep, the deflection of timber elements normally consists of two parts - an instantaneous deflection, and an additional long term deformation. However, due to the presence of a substantially stiffer element, the ASB, and the use of composite theory, it is thought sufficient to account for loss of stiffness of the timber over time when calculating the Young's Modulus.

$$E_{timber} = E_{d,sls} = \frac{E_{meanl}}{(1+K_{def})} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.18)$$

Where: K_{def} Deformation Modification Value

E_{mean} Mean Modulus of elasticity

$E_{d,SLS}$ Design Modulus of elasticity at serviceability limit state

Table 4.2 Deformation modification values k_{def}

Material	Standard	Service class		
		1	2	3
Solid timber	ES EN 14081-1	0.60	0.80	2.00
Glued Laminated timber	ES EN 14080	0.60	0.80	2.00
LVL	ES EN 14374, ES EN 14279	0.60	0.80	2.00
Plywood	ES EN 636			
	Part 1	0.80	–	–
	Part 2	0.80	1.00	–
	Part 3	0.80	1.00	2.50
OSB	ES EN 300			
	OSB/2	2.25	–	–
	OSB/3, OSB/4	1.50	2.25	–
Particleboard	ES EN 312			
	Part 4	2.25	–	–
	Part 5	2.25	3.00	–
	Part 6	1.50	–	–
	Part 7	1.50	2.25	–
Fibreboard, hard	ES EN 622-2			
	HB.LA	2.25	–	–
	HB.HLA1, HB.HLA2	2.25	3.00	–
Fibreboard, medium	ES EN 622-3			
	MBH.LA1, MBH.LA2	3.00	–	–
	MBH.HLS1, MBH.HLS2	3.00	4,00	–
Fibreboard, MDF	ES EN 622-5			
	MDF.LA	2.25	–	–
	MDF.HLS	2.25	3,00	–

4.1.6.2. Deflection limits

The recommended limits on deflection are given in Table 4.3. The limits on the absolute deflection will depend on the particular building construction, for example, the type of cladding.

Moment connections may be used to reduce serviceability deflections, although that is not covered in this publication.

Table 4.3 Recommended deflection limits for beams in general applications

Type of loading	Beam Type	
	Internal	Edge
Imposed loading	L/360	L/500
Imposed + cladding loading	-	L/360
Cladding loading only	-	L/500
Total deflection	L/200	L/250
Absolute deflection	50 mm	25 mm

4.1.6.3. Irreversible deformation

This check is required to ensure that yielding of the section does not occur at serviceability loads, so that the basic assumption that the beam remains linearly-elastic (made in calculating the deflections) is validated. The stresses in the bare steel section (arising from the self-weight loads in the construction condition) should be added to the subsequent stresses in the final composite condition (arising from the imposed loads, and any superimposed dead loads). For conservative reasons, the stresses are calculated assuming the concrete has cracked in the tension region of the cross-section at the composite stage. In accordance with BS 5950-3[12], the total steel stress should be less than, or equal to, p_y. A reduced section modulus may be used in stress calculations to allow for the interaction of local transverse bending, if considered appropriate. The concrete is limited to a stress of 0.5 f cu in compression.

4.1.6.4. Dynamic considerations

a. Natural frequency

When the individual structural components are inter-connected to form a complete floor system and the floor vibrates, the whole floor moves up and down in a particular form, known as a mode shape. Although, each floor frequency has a particular mode shape associated with it, it is generally the lowest (1st mode) or fundamental frequency that is of particular interest in design, owing to the fact that the largest acceleration response is normally found when this mode is excited from human activities. The fundamental frequency of the floor system is always lower than the frequency of any of the components.

Since primary beams are not normally required for floors comprising ASBs with CLT units, the calculation of the natural frequency is greatly simplified; this is because there is only one possible mode shape that may be sensibly considered, which is indicated in Figure 4.9. As can be seen from the figure, the mode shape is dominated by the ASBs vibrating as simply-supported members about the supporting columns. Owing to the fact that the floor slab flexibility is affected by the approximately equal deflections of the ASB (sinking) supports, the slab frequency is assessed on the basis that fixed-ended boundary conditions exist.

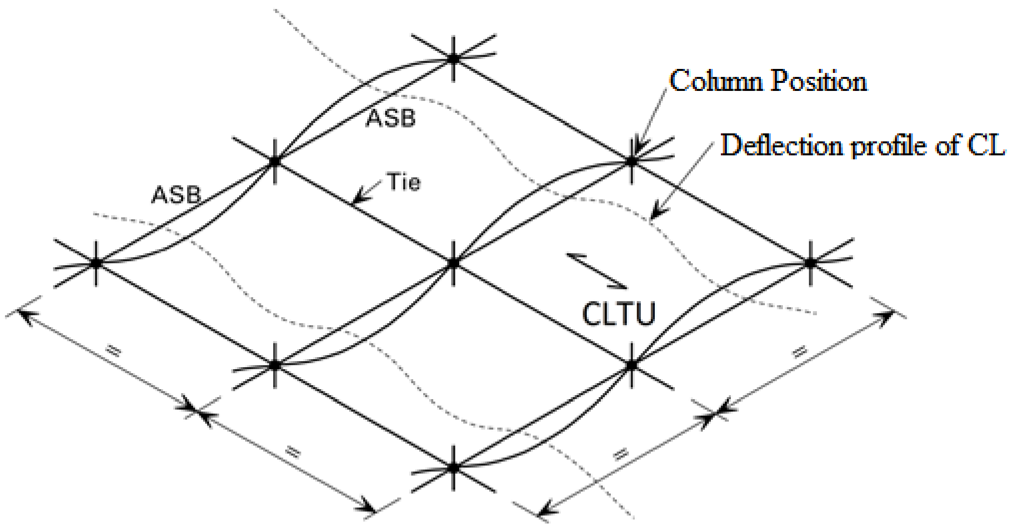


Figure 4.9 typical fundamental mode shapes for ASB and CLT unit floor system

The fundamental frequency of a complete floor f_0 may be calculated by summing the deflection calculated from the beam and slab components, and placing this value within Equation (3.21).

$$f_n = \frac{18}{\sqrt{\delta_{sw}}} H_z \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.19)$$

where δ_{sw} is the instantaneous deflection (in mm), owing to reapplication of the self-weight and other permanent loads acting on the beam, plus a proportion of the imposed load that may be considered permanent (an upper limit of 10% should be considered in the design of office and residential floors).

For cases when the floor grid is regular, the fundamental frequency of the floor may be evaluated by inserting, in Equation (3.21), the value for the instantaneous deflection given below:

$$\delta_{sw} = \frac{\omega b}{384E}\left(\frac{5L^4}{I_b} + \frac{b^3}{I_{slab}}\right) mm \ldots\ldots\ldots\ldots\ldots\ldots \quad (4.20)$$

Where ω is the load per unit area, b is the spacing of the ASBs, L is the span of the ASBs, I_b

is the untracked second moment of area of the composite ASB section (using the modular ratio values given in section 4.8.1) and I slab is the second moment of area of the CLT slab. Alternatively, it can sometimes be convenient to use these component frequencies directly, to evaluate the fundamental frequency of the floor f_0 by Dunkerly's approximation shown in Equation (3.23) below; both methods give the same results.

$$\frac{1}{f_0^2} = \frac{1}{f_1^2} + \frac{1}{f_2^2} + \frac{1}{f_3^2} \quad \ldots\ldots\ldots\ldots (4.21)$$

Where $f1$, f 2 and f 3 are the component frequencies (Hz) of the composite slab, secondary beams and primary beams (if any) respectively, with their appropriate boundary conditions, as defined above. For floors that are to be subjected to walking traffic, it is recommended that the fundamental frequency, f_0, should be at least 3.55 Hz. For floors that are to be subjected to synchronized crowd movement (such as used for aerobics, gymnasia, etc.), the effect of the dynamic loading on the Ultimate Limit State criteria should be considered. In accordance with BS 6399-1: 1996, resonant effects may be ignored if the fundamental frequency of the floor f_0 is greater than 8.4 Hz (based only on the self-weight and other permanent loads). If this frequency limit cannot be satisfied, the dynamic loads should be calculated directly using the method given in Annex A of BS 6399-1: 1996. In these circumstances, a partial factor of 1.0 should be applied to the dynamic loads and a partial factor of 1.4 to the dead loads.

4.1.7. Connection design

4.1.7.1. Introduction

Connections to ASB sections should normally be made using full depth end plates. The design of the connection should consider:

- The width of beams and column flanges.
- Requirements for torsional resistance (particularly for edge beams).
- Eccentricity of connection to suit beam alignments (particularly at edge beams)
- Requirements for sufficient bolts to resist shear and tension (when it is applied).
- The requirements for fillet welding.
- Extension of the end plate above the beam flange (and below for wind moment frames).
- Connections to RHS or CHS columns.

Standard dimensions have been adopted [18] to optimize these requirements. End plate connections to ASBs may be categorized [18] as either Flush Type A, Flush Type B or Extended. Flush Type A end plates are welded to the ASB around the inner profile of the ASB. Flush Type B end plates have an additional weld along the outer flange profile. Extended end plates overlap the ASB profile by a length sufficient for an extra row of bolts above (and below, if required) the section.

Where RHS or SHS columns are used, the use of Hollow-bolt or Flow-drill type connections may be used. Other connections may be used to suit the particular construction method but the designer must take into account the need for torsional rigidity of the connection. Detailed guidance on connection design is given in the following publications:

- Joints in Steel Construction: Simple Connections (P212)[19].
- Joints in Steel Construction: Moment Connections (P207)[20].
- Corus *Slimdek* Manual [18].

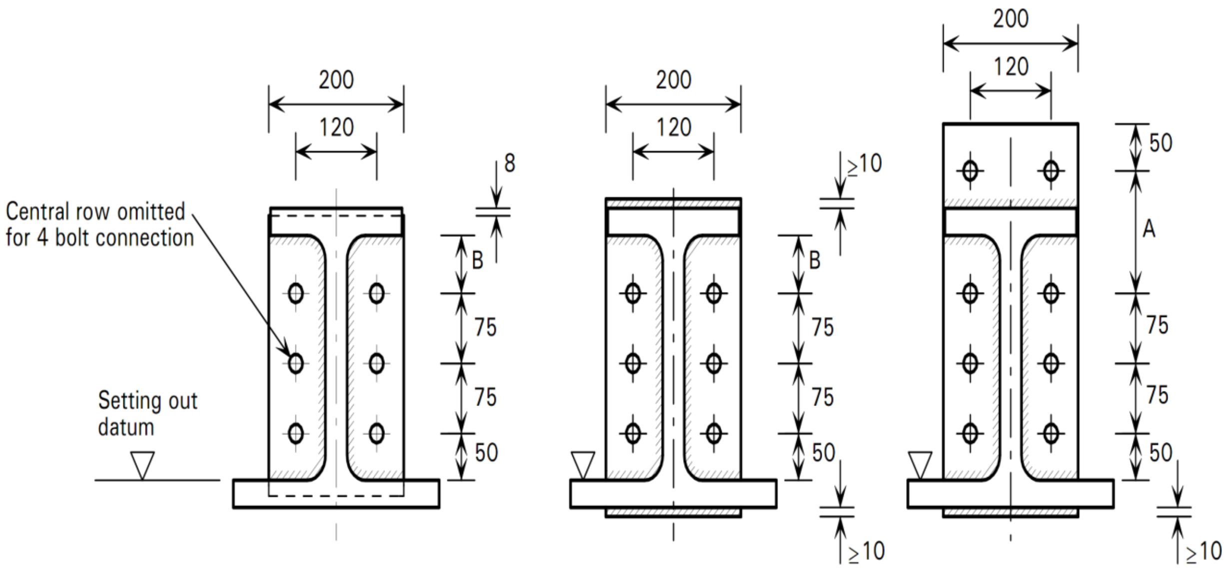
The design of the ASBs with moment end connections is not considered in this publication, although they could be used to improve the strength and stiffness of the beam.

4.1.7.2. Detailing

The setting out point for detailing of the connections is taken as the **top** of the bottom flange. This is done so that this level (which is the surface level of the slab minus the slab depth) is consistent for all ASB sizes. The lower bolts are positioned at 50 mm above the bottom flange. The recommended bolt-detailing rules are given in Figure 4.10.

The end plate should be a standard width of 200 mm for all ASB sections, which allows connections to 203 UC and larger columns. The bolt cross spacing should be 120 mm in order that the bolts are efficient in both tension and torsion. The vertical distance between the bolts should be 75 mm for 3 bolt rows and 150 mm for 2 bolt rows. These detailing rules differ from the SCI/BCSA 'Moment Connections' publication because of the thicker flanges of ASB sections in comparison to UB sections, and because of the shallower depth of section. These detailing rules provide connections that achieve sufficient shear resistance, bending resistance and stiffness. The recommended bolt size and end plate thickness for ASB connections is given in Table 4.4.

Dimension	A	B
ASB280	110	44
ASB300	140	62



(a) Flush end plate - Type A
(Shear and torsion only)

(b) Flush end plate - Type B

(c) Extended end plate

Figure 4.10 Recommended detailing dimensions for end plate connections to ASB sections

Table 4.4 Recommended bolt sizes and end plate thickness for ASB connections

	Grade 8.8 Bolt Diameter mm	End Plate Thickness (mm) Shear Resisting Connections	Moment Resisting Connections
Spans ≤ 6 m	20	10	12
Spans > 6 m	24	10	15

Modifications to these rules may be necessary for connection to edge beams. Such connections will require careful detailing because the beams are normally offset from the column center-line to suit the cladding.

4.1.7.3. Shear-resisting connections with torsional resistance - Flush Type A end plates to ASB sections

The normal method of connecting ASB beams to columns is to use a four or six bolt, full-depth flush end plate connection (see Figure 4.10(b)). These connections possess excellent shear and torsional resistance which is utilized at the construction stage, or where the beam is subjected to high out of balance forces, for example, edge beams.

4.1.7.4. Moment-resisting/shear resisting connections - Flush type B or extended end plates to ASB sections

These types of connection are suitable for resisting pure shear, or combined shear and moments. They generally require the use of a thicker end plate fully welded to the ASB flanges. Extended end plates using 8 bolts can develop end moments of at least 10% of the moment capacity of the beam. Moment resistance and shear and torsion capacities based on the standard end plate details and minimum weld sizes are given in the Corus *Slimdek* Manual. They have been developed from the guidance given in the SCI/BCSA publication P207.

4.1.8. Robustness

Robustness of structures relates to the resistance to accidental damage and unusual loadings, such as explosions. There is a statutory requirement for avoidance of "disproportionate collapse" of buildings. Codes cover this requirement by specifying minimum tying forces between the various elements. A steel framed structure achieves tying resistance by appropriate design of the beam-to-column connections. In general, the following tying systems are required:

- ✓ Peripheral ties around the perimeter of the building.
- ✓ Internal ties between the internal beam and floor slab.
- ✓ Internal ties between the columns (may be distributed across the slab).

The measures required for diaphragm action and fire resistance (which are discussed in the next sections), normally achieve sufficient robustness of the construction.

CHAPTER FIVE

5.1. Comparison of Asymmetric Slimflor beam and cross laminated timber composite floor with concrete solid slabs

5.1.1. Foundation Load Comparison

Loads to columns and foundations are shown in chart 5.1 to 5.3. The values are calculated by multiplying the area of the bay by the design load, comprising of the slab self-weight and the imposed load. The steel used in each bay is then added. The use of this value as a column load assumes identical bays in all four quadrants around the column. What is apparent from the data is that using XLT slab produces a marked reduction in axial loads to columns (and therefore loads to foundations), ranging between 14% and 60%. On average, axial load reduction is 48%. Smaller axial loads lead to a reduction in the size of foundations needed, and because of the lower forces coming from each bay, overturning moments from any asymmetric bay sizes around the column will also be reduced. Reduced loads mean reduced reinforcement is needed, saving costs in material, labour and design.

Foundations represent between 6 % and 10% of a structure's total cost, so this level of reduction would be of significant benefit. Foundations can be complicated by having to negotiate existing groundwork’s, pipes etc., so reducing their size is also beneficial in terms of ease of design. [3]

Chart 5.1 Foundation Load, 6m x 9m grid

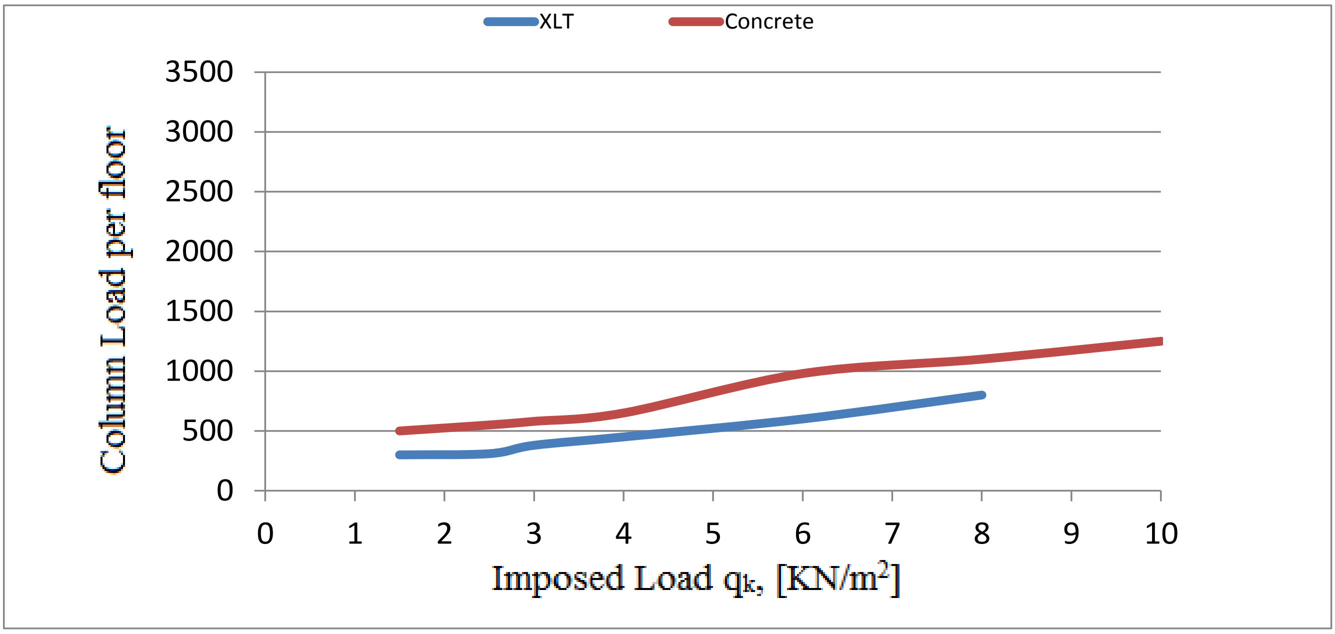


Chart 5.2 Foundation Load, 9m x 9m grid

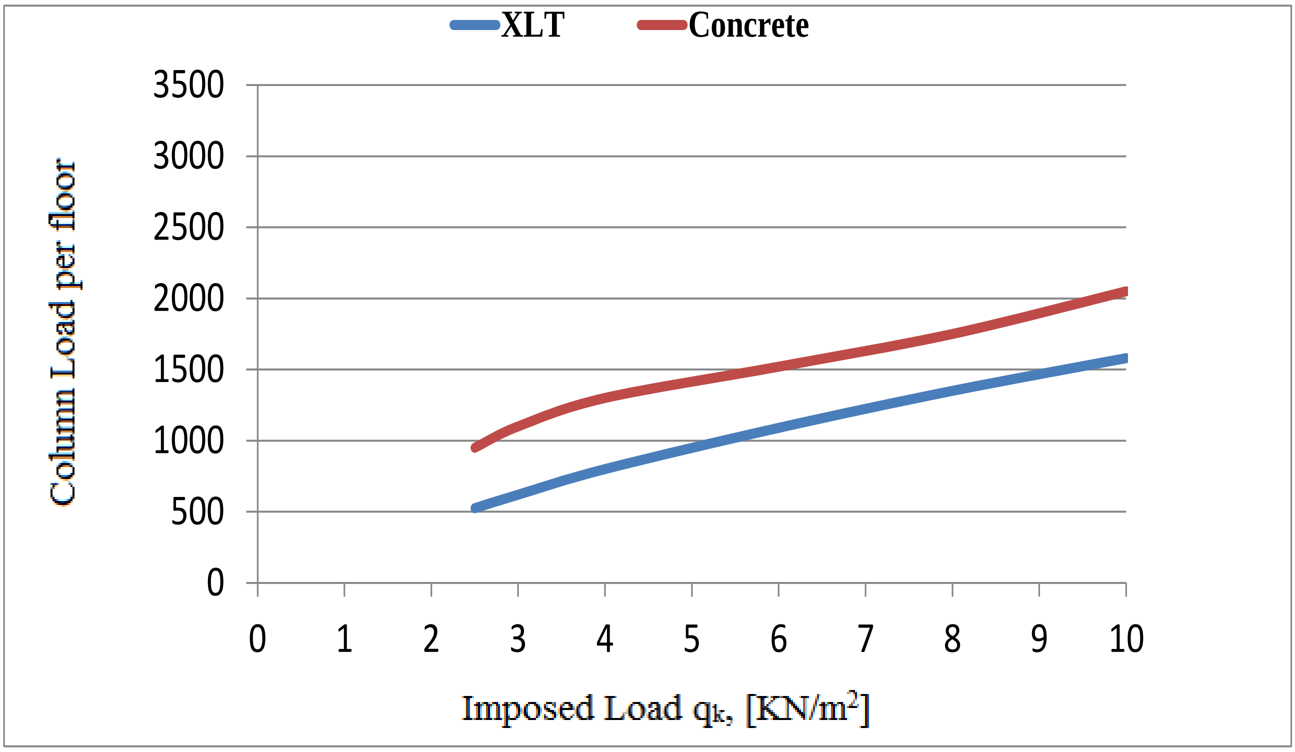
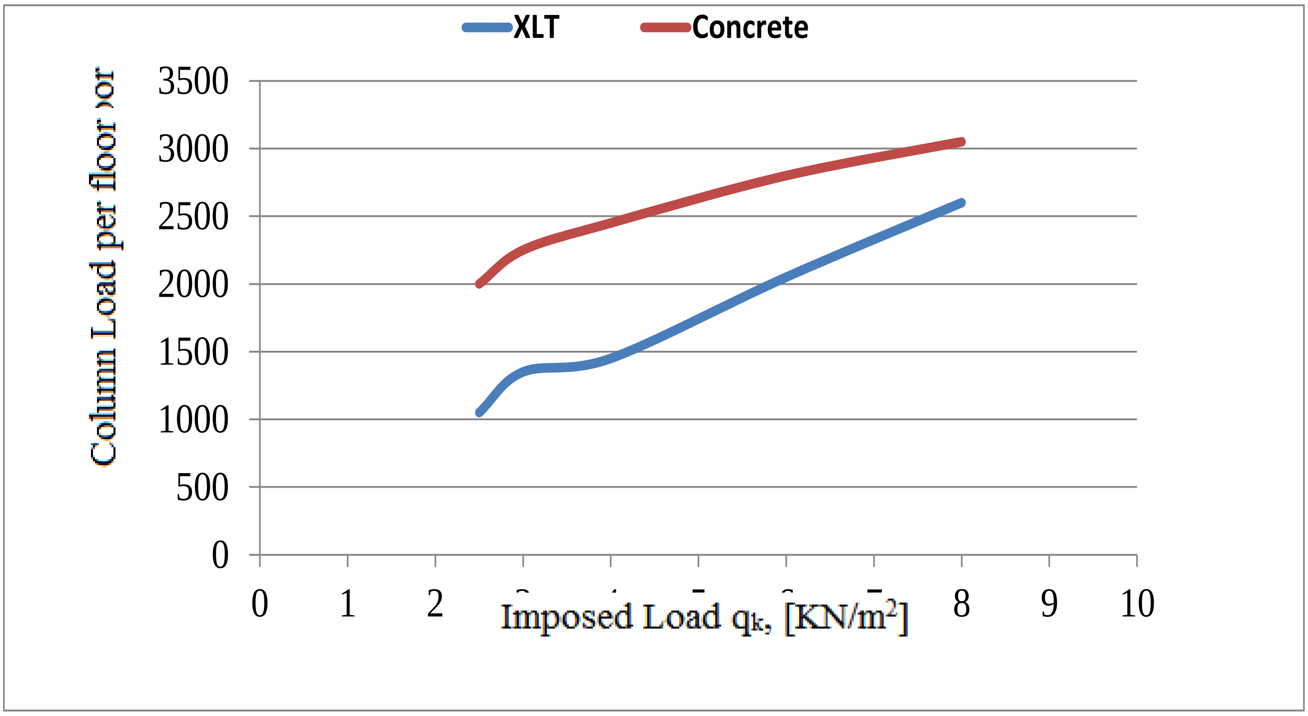


Chart 5.3 Foundation Load, 12m x 12m grid



5.1.1.1. Impact on foundation volume

A method of approximating the reduction of the foundation size by using timber has been derived. This assumes a pad foundation for comparison, with no moment in either axis.

To prevent failure in the soil, the limiting requirement is set by

$$\sigma_{ABP} = \frac{N}{A_{min}} \, \alpha \, \frac{N}{d2_{min}} \therefore d2_{min} \, \alpha \, N\sigma_{ABP} \ldots\ldots (5.1)$$

Where

σ_{ABP} = Allowable Bearing Pressure of the soil

N = Axial load from the column

A_{min} = Minimum bearing area

d_{min} = Minimum dimension of bearing area

This means that if the axial force in the column is reduced by using CLT relative to concrete, by introducing a scale factor, λ, then the reduced bearing area dimension is found as follows.

$$N_{XLT} = \lambda \, N_{CONC} \ldots\ldots (5.2)$$

$$\rightarrow \quad d_{min,XLT}^{2} = \lambda \, d_{min,CONC}^{2} \ldots\ldots (5.3)$$

$$\rightarrow \quad d_{min,XLT} = \lambda \, 1/2 \, d_{min,CONC.} \ldots\ldots (5.4)$$

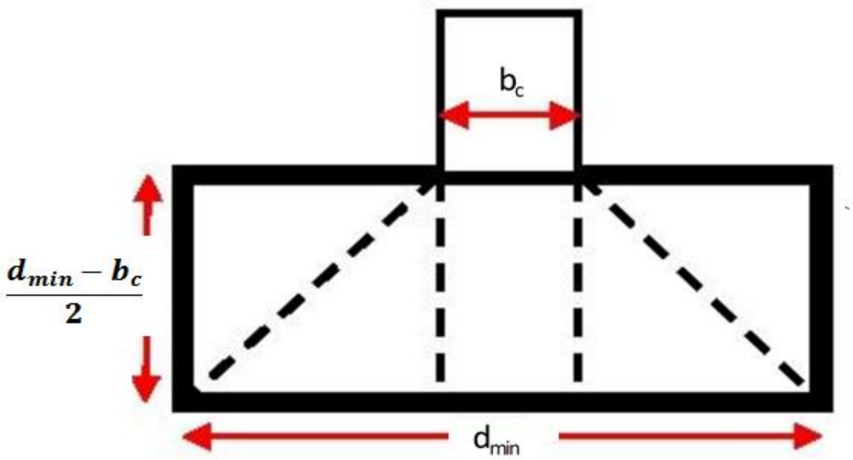


Figure 5.1 Nominal pad foundation

Transmission of stresses from the edge of the column to the bottom of foundations occurs at a 45° angle. Geometry necessitates a minimum pad depth of $(d_{min}-b_c)/2$, where bc is the breadth of the column. For simplicity of the approximation, minimum pad depth will be taken as $d_{min}/2$. This is shown in Figure 5.1.

$$Vol_{Founds} = \frac{d_{min^3}}{2} \ldots\ldots (5.5)$$

Substituting parameters from (5.4),

$$Vol_{Founds,XLT} = \frac{d_{min,XLT^3}}{2} = \frac{(\lambda^{1/2} d_{min,CONC})^3}{2} = \lambda^{3/2}.\frac{d_{dmin,CONC^3}}{2}$$

$$Vol_{Founds,XLT} = \lambda^{3/2} V_{Founds,CONC}$$

Therefore, when the column loads with a concrete slab are reduced by a factor of 0.5, the foundation volume using timber will be reduced to approximately $\mathbf{0.5^{3/2}}$ **(=0.35)** times the size. This ratio will be used in the cost comparison section.

5.1.2. Cost Analysis

i. Methodology

The scope of the cost analysis is limited to the contribution of the structural framework and foundations to the overall cost of the structure. The cases analyzed are only those where the XLT solution has been shown to be more economical in terms of steel usage. Using the steel usage data and the floor areas from section 5.1, the cost of the structural framework can be approximated using material costs. The overall cost of constructing the superstructure can then be derived. A relative cost of foundations compared to superstructure yields a cost ratio that is applied to the concrete slab case. Finally, the assumption is made that foundation cost is proportional to foundation volume - the reduction in foundation cost through use of concrete can then be found from by multiplying by $\boldsymbol{\lambda}^{3/2}$, as explained in section 5.1.[3]

ii. Parameters

The costs of materials used in the analysis are presented in table 5.1

Table 5.1 Material Costs for 6*9m bay slab

ASBs Vs XLT COMPOSITE FLOOR MATERIAL COSTS				
COMPONENT	INDICATED PRICE	SOURCE	DATE OF PRICE	ASSUMED PRICE
ASBs	EBR28,080 per tone	TATA[12]	2004	EBR28,100 per tone
Timber Panels	EBR69,984per bay	http://con.2merkato.com	Apr-06-2017	EBR70,000per bay
Surface finish paint	EBR1650per bay	http://con.2merkato.com	Apr-06-2017	EBR1,700per bay
CONCRETE SLAB AND ITS FINISHING MATERIAL COSTS				
COMPONENT	INDICATED PRICE	SOURCE	DATE OF PRICE	ASSUMED PRICE
Concrete (slab beam)	EBR35,868per bay	http://con.2merkato.com	Apr-06-2017	EBR36,000per bay
Reinforcement (slab beam)	EBR47,844 per bay	http://con.2merkato.com	Apr-06-2017	EBR48,000 per bay
Ceramic tilling	EBR18,900per bay	http://con.2merkato.com	Apr-06-2017	EBR19,000per bay
Cement mortar	EBR5,940per bay	http://con.2merkato.com	Apr-06-2017	EBR6,000per bay
Scaffolding & Formwork	EBR18,900per bay	http://con.2merkato.com	Apr-06-2017	EBR19,000per bay

5.1.2.1. Material - Total superstructure scale factor

According to the Steel Designer's Manual [64], "Material costs represent only 30-40% of the total cost of structural steelwork" Taking 35% as the material percentage, superstructure cost, CSS, is given by

$$C_{ss} = \frac{C_{mat}}{0.35}$$

Where C_{mat} is total cost of material

5.1.2.2. Superstructure - foundation scale factor

From cost models developed by Davis Langdon, it is conservatively assumed that the foundations make up approximately 6.5% of the total building cost, while the superstructure constitutes 10%. Hence $C_{Founds} = 0.65C_{ss}$

Where C_{Founds} is the cost of foundations

Therefore the transformation factor from material cost, C_{mat}, to foundation cost, C_{Founds}, is 0.65/0.35 = 1.86.

$$C_{Founds} = 1.86C_{mat}$$

N.B. This conversion factor applies to the concrete slab solution only. Foundation costs for the XLT solution are derived from the equivalent concrete case and multiplying by the factor $\boldsymbol{\lambda}^{\mathbf{3/2}}$, where $\boldsymbol{\lambda}$ is given by

$$\boldsymbol{\lambda} = \frac{N_{XLT}}{N_{CONC}}$$

Chart 5.4 shows a comparison of the main costs that vary with the choice of slab material. From the figures, it is clear that the XLT-ASB hybrid structure is the expensive option when we see only the floor to floor comparison. Using XLT slabs gives cost savings in the **steel frame, Finishing, formworks** and substantial savings in the **foundations**, but these savings are offset by the sizeable premium in the cost of the slab. As can be seen in table 5.1 the timber slabs are over $1^1/_2$ times as costly as solid concrete slabs, becoming the most significant contributor to overall cost in place of the steel frame. If slab costs can be reduced, (in my calculation I take for local material timber price) the system could be much more of a viable alternative to conventional building techniques.

Chart 5.4 Cost (Per Bay) break down and comparison (6mX9m)

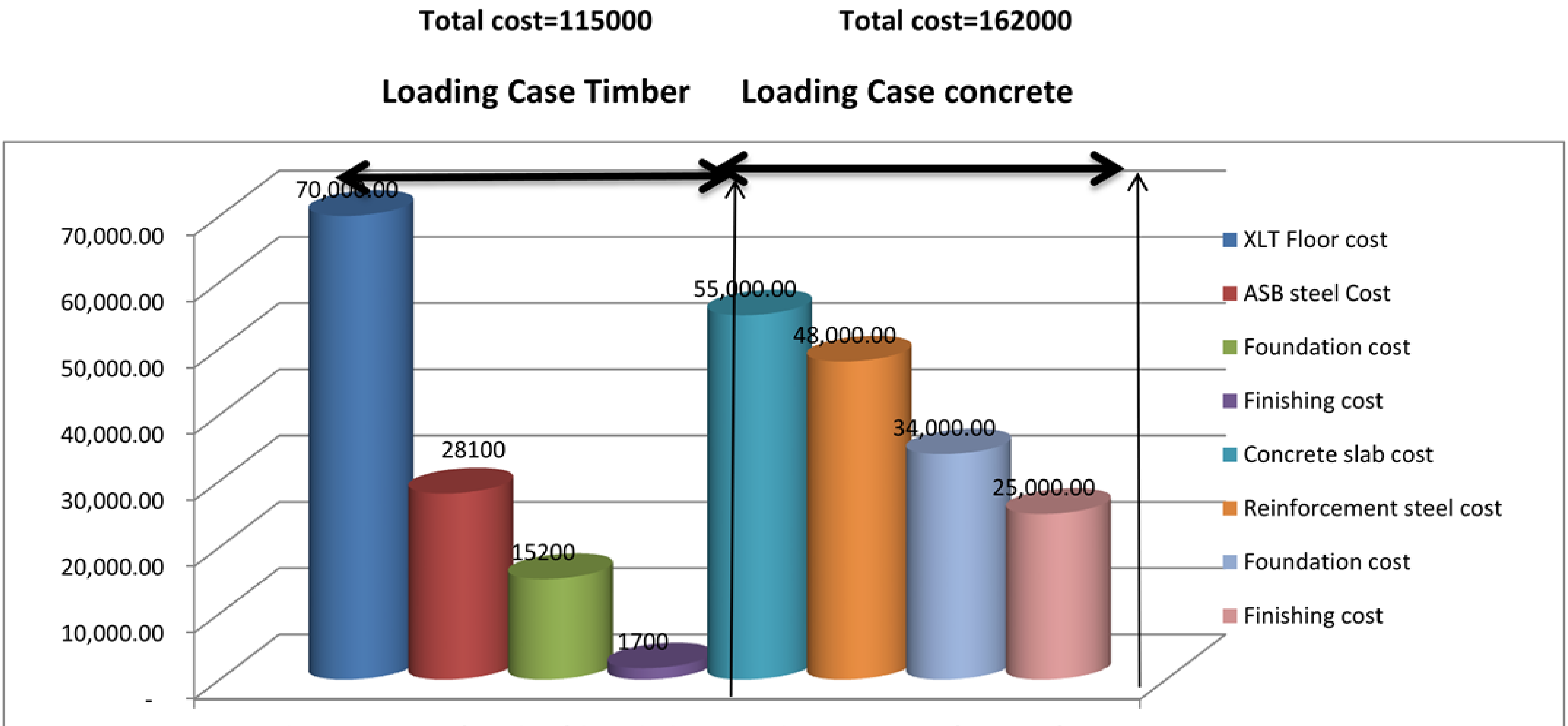


Chart 5. 4 cost (per bay) break down and comparison (6mx9m)

With limitations on time and the degree of detail it is possible to go into, this cost model gives only a general sense of the contributory factors to cost. There are other aspects that it does not consider, such as duration of works due to ease of construction, labour costs and transport costs. In addition, the derivation of foundation costs makes liberal approximations that may not accurately reflect the real cost. The costs used for the model are also not as accurate as could be desired due to the absence of readily available and up to date data. For the timber case evidently the cost of the slab is a critical determiner of overall cost; however the suppliers of the XLT slabs were not forthcoming with information on their pricing so the value quoted is for generic engineered laminated timber panels.

With these shortcomings, the model most likely will not accurately predict the cost of a building using the timber or concrete slabs, however it does allow comparisons to be drawn between the systems and demonstrates the critical characteristics.

A more detailed cost analysis and comparison may give different conclusions to the one found here, and would beneficial for clarity on the issue. Cost is a key factor when deciding the structural form of a structure meaning unless the cost effectiveness of the XLT slabs can be proven or enhanced, by farther research's it may not be considered as an option.

5.1.3. Disassembly and Reuse

5.1.3.1. Steel-Timber vs. Concrete – An Environmental Comparison

A research project entitled Embodied carbon in natural building stone in Scotland' by Dr. Suzy Goodsir and Naeeda Crishna found that substituting concrete for timber gave significantly reduced CO2 emissions associated with the structure, and that for this reason, timber as a building material becomes much more competitive when CO2 emissions are costly, and modeling suggests demand for timber would increase accordingly. For systems with comparable heating and cooling requirements wood-based building systems generally contain lower embodied energy and CO2 emissions than steel, concrete, and brick-based systems.

The extent of the environmental benefit of timber is of course limited. If there was a shift towards timber as a building material, there would need to be an increase in the amount of forestry that is managed for sustainable growth and high yield. From a recycling perspective, it is found that the recycling and reuse of timber building products is more beneficial than recycling concrete. After primary use, concrete can be recycled by crushing for use as aggregate, but due to the recycling process, compared to fresh aggregate it is difficult to use and would corrode reinforcing steel, thus limiting the level of quality that can be achieved and the range of applications. Timber is currently either reprocessed into new wood based products or burned as a biofuel. Whilst combustion of wood releases CO2, it is better environmentally than fossil fuels because during the trees life, it absorbed CO2 during photosynthesis and stored it for many years, and is also a renewable source of energy. The table below shows the levels of CO2 in various building materials.

Table 5.2 CO2 amounts in building materials in kgCO2/ton

Building materials	**$kgCO_2$ /tonne**
Sandstone	64
Granite	93
Marble	112
General Concrete	130
General Clay Bricks	220
Slate	232
Timber	450
Facing Bricks	520
General Building Cement	830
Steel: Bar and Rod	1710
Steel: Galvanised sheet	2820

Source: Project by Dr Suzy Goodsir and Naeeda Crishna from Scotland

This table clearly shows that timber contains less kg CO2 per ton as compared to what is contained in general building cement and in steel bar and rods which make the reinforced concrete used in normal construction. [10]

5.1.3.2. Reuse of Timber

The environmental benefit of using timber slabs rather than concrete depends on the ability of the slabs to be reused at the end of the structure's life, which in turn depends on how durable the slabs can be. As discussed above, there would be additional strain put on forest reserves if there were a increased demand for timber as may result from the widespread uptake of the proposed system. However the more modest usage of timber in this system reduces the magnitude of the additional strain put on forest reserves than it is supposed to be. Maximizing the working life (and hence the reusability) of the timber slabs would help further in this issue as there would be less need for fresh specimens.

The timber slab can be specified to have surfaces that meet visual quality specifications, and can be used as a working surface. Nevertheless, to improve the recycling potential, treatments are available to protect the timber from deterioration. To guard against fungal and other biological attack, there are a variety of preservative types, including water borne, micro emulsion, and organic solvent preservatives. There are also many different ways of applying the preservatives, depending on the properties of the wood being preserved and the final use e.g. immersion, spraying, or most effectively, pressure impregnation.

The slabs will also need to be sealed against water infiltration. The edges of the slabs will be particularly susceptible because the end grains are located there and the panel will absorb moisture more readily in this direction due to the microstructure of the material.

Absorption of moisture can cause swelling, discoloration, and decay. The swelling will be a particular issue because it will reverse some of the processing that went into making the board meaning that even if the slab were to dry out, the original shape would not be regained. Due to the configuration of the flooring system, the ends and sides of the slab units will be inaccessible for maintenance, meaning sealing will have to be done to a very high standard, and measures must be taken to avoid excessive water contact with the slab.

5.1.4. Aesthetic value and constructability

5.1.4.1. Aesthetic value

Timber construction has real appeal to a small but probably growing proportion of building clients and design professionals. They appreciate the aesthetical properties of visual grade glulam timber, appearance hardwoods and engineered wood products, the improved internal conditions (acoustics and warmth) found in wood rich environments, the high strength to weight structural properties of the material, and its resistance to corrosive environments.

5.1.4.2. Constructability

To facilitate the placement of the slabs during construction, notches may need to be built into the XLT to give clearance to the top flange of the ASB. However, this has implications for the shear capacity of the section - if notches are employed then only the reduced section depth shall be assumed to be acting in shear. The following are the advantageous of using timber steel hybrid floor over concrete floor slab on the construction phase.

- Quick erection times
- Reduced site labour
- Reduced time to weather the structure
- Earlier introduction of following trades
- Low embodied energy if constructed in local timber
- Recyclable
- Reduced construction waste through efficient controlled manufacturing
- Low volume of waste on site requiring removal
- Can be built to exceed 60-year design life
- Energy efficient when constructed to current standards
- Fast heating due to low thermal mass
- Reduced time on site reduces environmental nuisance and disruption to local residents
- Engineered product
- Factory controlled quality assurance in fabrication
- Efficient use of material due to controlled engineering and fabrication
- Reduced construction time translates into reduced risk exposure

CHAPTER SIX

6.1. Conclusions and Recommendations

6.1.1. Conclusions

This study has found the construction of Asymmetric slimflor beam with cross laminated timber composite flooring related to how far the design is capable for the given building floor area in terms of ultimate limit state and serviceability limit state design criterion requirements. It is well known that in developing countries like Ethiopian construction industry is facing to reinforced concrete flooring. But, this study found the major advantage of using Asymmetric slimflor beam with cross laminated timber composite flooring over concrete flooring are ; even if the precast concrete structural units are easy to construct however, steel timber floor is more easier to construct or installation in short period of contract time with good quality since it is not depend on false works like concrete, Reused and changeable , since steel timber is reduce the total weigh of the entire building the foundation volume and size is reduced, us the analysis of this study the material cost considered specifically per floor area of the timber cost is two times expensive than concrete floor but, from the whole building due to lighter load and low steel usage timber steel flooring is 30% low cost than concrete flooring and environmentally friendly than concrete flooring those whole issues are addressed in this study.

6.1.2. Recommendations

1. Furthermore this study aim to apply and introduce the issue of design and analysis with construction of Asymmetric slimflor beam with crosses laminated timber composite flooring in construction sector of the country. This can do after further proof of this study.
2. It is well known that the current world's biggest issue is the carbon emission which creates environmental crisis. Where us if this study is practically applied;

- ✓ The rural population with excess land may plant the trees for wood industries and it is able to be the source of business for our poor farmers (poverty reduction)
- ✓ Today the planted trees may be deforested for different purposes at their teenage but, for wood flooring the wood may attain to its size and strength whenever it becomes big enough it keeps environment balanced against absorbing carbon. (Environmental)

❖ In Ethiopia flow of population from rural to urban attains high rate, this cause the cities densely populated, which create housing crisis. Hence if this study is applied the housing problem is resolved in short period of time and low cost of construction.

- Further research should be conducted on connection of ASB with CLT composite floor action.
- Design of ASB with CLT should be checked against diaphragm action.

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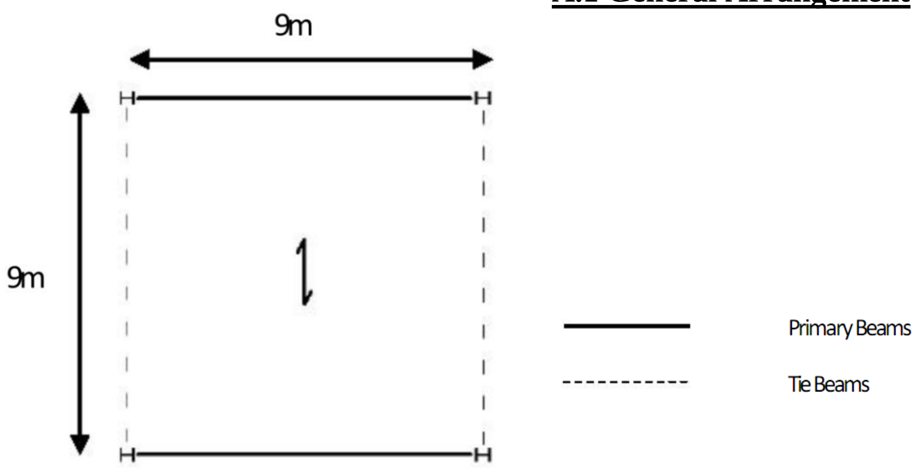
APPENDIX -A

	XLT slab in 9m x 9m grid Office	**Page1 of 16**

WORKED EXAMPLES

Here are presented the verification process for slabs and beams for the timber slab. For this worked example, the bay size is 9m x 9m and the loading is for an Office.

A.1 General Arrangement



Imposed Load, qk : Office (Use category B1) $2.50 KN/m^2$

Superimposed Permanent Load: $0.50 KN/m^2$

Properties

Slab designation	KLH 7ss260	
Slab depth, d	260mm	
Density, ρ_{XLT}	480kg/m3	4.7kN/m3
Span, I	9m	
Slab unit width, b	2.5m	
Nominal bearing length, l_b	80mm	

Slab self-weight, $g_k = d.\,\rho_{\text{XLT}} + 0.5$ = 4.7 x (260/1000) + 0.5 = 1.72kN/m2

Design Loads

$\gamma_G = 1.35;\ \xi = 0.925;\ \gamma_{Q1} = 1.5; = \psi_0 = 0.70$

$f_{d.1} = \gamma_G.g_k + \gamma_{Q1}.\psi_0.q_k = (1.35 * 1.72) + (1.5 * 0.7 * 2.5) = \mathbf{4.97 KN/m^2}$

$f_{d.2} = \xi.\gamma_G.g_k + \gamma_{Q1}.q_k = (0.925 * 1.35 * 1.72) + (1.5 * 2.5) = \mathbf{5.90 KN/m^2}$

A.2 SLAB BEARING CHECK

Design End shear force, V_d $=\frac{f_{d,max}.b.l}{2} = \frac{5.90*2.5*9}{2} = 66.4KN$

Design Bearing stress, $\sigma_{c.90.d}$ $=\frac{V_d}{b.l_b} = \frac{66.40*10^3}{2500*80} = 0.33N/mm^2$

Design Bearing strength, $f_{c.90.d}$ $= \frac{k_{mod.p}.k_{sys.}f_{c,90.k}}{\gamma_M}$ From Eq. (3.3)

$k_{mod.p} = 0.60 \quad (Service\ class\ 1\ or\ 2, Permanent\ action)$

$k_{sys} = 1.00$

$\gamma_M = 1.00$

$f_{c,90.k} = 0.60$

$$f_{c,90.k} = \frac{0.6 * 1.00 * 2.5}{1.25} = 1.20N/mm^2$$

As $\sigma_{c.90.d}$ (= 0.33) < $f_{c.90.d}$ (= 1.2), the slab is verified in bearing.

Allowance for non-rigid supports

$$\tau_{v.d} = \frac{3}{2}\frac{V}{K_{cr}bh}$$ From Eq (3.2)

Design shear stress, $\tau_{v.d} = \frac{3}{2}\frac{V}{K_{cr}bh} = \frac{3}{2}\frac{66.40*10^3}{0.65*2500*260} = 0.228/mm^2$ Where, $K_{cr} = 0.67$

Design shear strength, $f_{v.d} = \frac{k_{mod.p}.k_{sys.}f_{v.g.k}}{\gamma_M}$, Where, $f_{v.g.k} = 2.70N/mm^2$ From table. (3.2)

$$f_{v.d} = \frac{0.6 * 1.0 * 2.7}{1.25} \mathbf{1.30N/mm^2}$$

$0.35f_{v.d}$ (= 0.455 N/mm^2) > $\tau_{v.d}$ therefore support stiffness does not need consideration.

A .3 FLOOR FIRE PERFORMANCE (ASSUMING 1M STRIP)

XLT flexural strength, $f_{m.k}$ = 24 N/mm2 From table. (3.2)

A.3.1 Structural Failure - Janssens' method (normal loading condition)

Design UDL , w_d $= f_{d.max} * 1m = 5.90\boldsymbol{KN/m}$

Design Bending Moment, M* $= \frac{w_d.l_b{}^2}{8} = \frac{5.90*\mathbf{9^2}}{8} = 59.70\boldsymbol{KN/m}$

Section Modulus, Z $= \frac{b.d^2}{6} = \frac{1*\mathbf{0.26^2}}{6} = 0.0113\boldsymbol{m^3}$

Design Moment Resistance, Mn $= \frac{k_{mod.p} \cdot f_{m.k}}{\gamma_M} . Z = = \frac{0.6*0.0113*24}{1.25} * 1000 = 130.20 KN.m$

Load Ratio, Rn $= \frac{M^*}{M_n} = \frac{59.70}{130.20} = 0.459$

Time to structural failure, t_{sf} = $1.25d\left(1 - \sqrt{0.4R_A}\right) - 11.3$ From Equation (3.7) of this thesis

$$1.25x260 * \left(1 - \sqrt{0.40.459}\right) - 11.3 = 186 mins$$

A.3.2 Eurocode Method - Reduced properties strength verification at 90 minutes

XLT Char rate, β = 0.76 mm/min

Perimeter exposed to fire, p = 1m

Fire load UDL, $wd_{,f}$ = $(g_k+0.5q_k)$ *1m = 1.72+(0.5*2.5) = 2.97KN/m

Design Bending Moment, M^*_{fire} = $\frac{w_{d.f} l_b{}^2}{8}$ = $\frac{2.97*9^2}{8}$ = $30.10 \boldsymbol{KN/m}$

Depth of char (@t=90),c = $\beta.t = 0.76 * 90 = 68.40 mm$

Reduced section Depth, d_f = d-c = 260-68.40 = 191.60mm

Reduced section Modulus, z_f = $\frac{b.d_f{}^2}{6}$ = $\frac{1*0.1916^2}{6}$ = $0.0061 m^3$

Reduced section Area, A_f $= bd_f$ = $1 * 0.1916$ = $0.192 m^2$

Design Strength of Remaining Section, $M_f = K_{mod.fi} . \frac{k_{fi} \cdot f_{m.k.} Z_f}{\gamma_{M.fi}}$ From Eqn. (3.9) of this thesis

$$k_{mod.fi} = 1 - \frac{1}{200} \frac{p}{A_r} = 0.974$$

$$k_{fi} = 1.15$$

$\gamma_{M.fi}$ = 1.00 , M_f = ***164.5KNm***

As Mf > M*fire, the slab is verified for strength at 90mins fire resistance

A.3.3 Eurocode Method - Time to integrity failure

Notional charring rate, β_n =0.7 mm/min

Time to integrity failure, t_{if} = $\frac{\xi d}{\beta n}$ For $\xi = 0.2, \quad t_{if} = 74 mins$ ***Fails***

$\xi = 0.3, \quad t_{if} = 111 mins$ ***Ok***

Therefore, to prevent integrity failure, stipulate lap joints between XLT slabs, with lap dimension greater than 30mm

A .4 BEAM VERIFICATIONS

This verification is based on the methods presented in SCI document P342, with changes to fit the construction scheme and material usage discussed.

Primary Beam: 300 ASB 196

A.4.1 Beam Properties

Elastic N.A. depth	y_e	198 mm
2nd Moment of Area (Major Axis)	I_Y	45,871 cm4
2nd Moment of Area (Minor Axis)	I_Z	10,463cm2
Radius of gyration	r_y	6.5mm
Torsional Constant	I_T	1177cm4
Elastic Modulus (top)	$W_{el,y}$	2321cm3
Plastic Modulus	$W_{pl,y}$	3055cm3
Cross Section Area	A	249cm2
Warping Constant	I_w	1,500,000cm6
Buckling parameter	u	0.845
Torsional Index	x	7.86
Flange width (top)	B_t	183mm
Flange Width (bottom)	B_b	293mm
Depth of Section	D	342mm
Web thickness	t_w	20mm
Flange thickness	t_f	40mm
Root Radius	r	27mm

Section is Class 1

Beam Spanning distance 9m

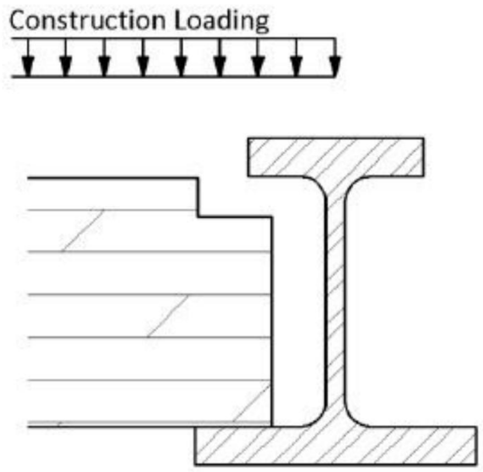
Beam Spacing (Slab span) 9m

Beam weight = 196kg/m =1.92KN/m

A.4.2 Construction Stage

Imposed Load = Construction Load = 0.5 kN/m2

Case 1: (XLT slab + Construction Loading), on one side of ASB only.



Assume bearing length of 40mm for design

Imposed UDL, qk = 0.5 x 4.5m = 2.25 kN/m

Permanent Load (slab) = Slab unit weight = 4.7 x (260/1000) = 1.22 kN/m2

Permanent Load (beam) = 1.92 kN/m

Permanent UDL, gk = slab + beam = (1.22 x 4.5m) + 1.92 = 7.41 kN/m

$f_{d.1} = \gamma_G.g_k + \gamma_{Q1}.\ \psi_0.q_k$ = (1.35 x 7.41) + (1.5 x 0.7 x 2.25) = 12.4 kN/m

$f_{d.2} = \xi\ \gamma_G.g_k + \gamma_{Q1}.q_k$ = (0.925 x 1.35 x 7.41) + (1.5 x 2.25) = 12.6 kN/m

f_d = max($f_{d.1}$; $f_{d.2}$) = **12.6 kN/m**

Lateral Torsional Buckling (LTB) for combined bending and tension

Verify against interaction formula:

$$\frac{M_y m_{LT}}{M_y m_{LT}} + \left\{\frac{\sigma_{bzT}+\sigma_w}{f_y}\right\}\left\{1 + \frac{0.5.M_y m_{LT}}{M_{b,rd}}\right\} \leq 1$$

From Equation (4.6) of this thesis

Major axis moment, M_y $\frac{f_d.l^2}{8} = \frac{12.6*9^2}{8} = 128\boldsymbol{KNm}$

Torsion Load $4.5m * 9m * [(0.925 * 1.35 * 1.22) + (1.5 * 0.5)] = 92.10KN$

Torsion Lever arm $= \frac{B_b - 40}{2} = \frac{293-40}{2}$ 126.50mm

Applied torsion, Tq $= 92.10 * (126.50/10^3$ 11.70KNm

Finding buckling resistance of the section, $M_{b,Rd}$:

First, elastic critical moment (M_{cr}) is given by $M_{cr} = C_1 \frac{\pi^2 EI_Z}{L_{cr}^2}\left[\frac{I_w}{I_z} + \frac{L_{cr}^2 GI_T}{\pi^2 EI_Z}\right]^{0.5}$

c1 = 1.132 (Simply supported beam under UDL)

Lcr = 9m (Beam Assumed unrestrained during construction)

$$M_{cr} = 1.132 \frac{\pi^2 * (210 * 10^9 * \frac{10,463}{10^8}}{L_{cr}^2} * \left[\frac{\frac{1,500,000}{10^{12}}}{\frac{10,463}{10^8}} + \frac{9^2 * (81 * 10^9)(\frac{1,177}{10^8})}{\pi^2(210 * 10^9) * \frac{10,463}{10^8}}\right]^{0.5}$$

$$= 1,844,561 Nm = 1,845 KN.m$$

Buckling parameter, $\lambda_{LT}$is then given by $\lambda_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$ From Equation (4.12) of this thesis

W_y = $W_{pl,y}$ = As section in class 1

f_y = 355Mpa $\lambda_{LT} = \sqrt{\frac{3055*355}{1845*1000}} = 0.767$

$$\frac{D}{B_t} = \frac{342}{183} = 1.87 < 2 \ therefore\ imperfection\ factor\ \alpha_{LT} = 0.21$$

Now, reduction factor for LTB, χ_{LT} is given by

$$\chi_{LT} = \frac{1}{\phi_{LT[\phi_{LT}{}^2 + \lambda_{LT}{}^2]}^{0.5}} \leq 1.00$$

Here, $\phi_{LT=0.5[1+\alpha_{LT}(\lambda_{LT}-0.2)+\lambda_{LT}{}^2]} = 0.853$

Therefore,

$$\chi_{LT} = \frac{1}{0.853 + [0.853^2 - 0.767^2]^{0.5}} = 0.814$$

Finally, design buckling resistance of unrestrained beam, $M_{b,Rd}$ is calculated as

$$\boldsymbol{M_{b,Rd} = \chi_{LT} W_{pl,y} \frac{f_y}{\gamma_M} = 814 * 3055 * 355 = 882,803 Nm = 883 KNm}$$

Torsional Parameters:

Torsional Bending constant, a, is given by

$$a = \left(\frac{EI_W}{GI_T}\right)^{0.5} \ From\ Eqn. (4.11) of\ this\ thesis = \left(\frac{210 * (1,500,000 * 10^6)}{81 * (1177 * 10^4}\right)^{0.5} = 575 mm$$

$$\frac{L}{a} = \frac{9000}{575} = 15.70$$

Using L/a value and Table 4.1 of this thesis, by linear interpolation:

$$\frac{\varphi G I_T}{T_q a} = 1.9 \quad \Rightarrow \quad \varphi = 0.0124 \ Radians$$

and

$$-\left(\frac{\varphi'' G I_T a}{T_q}\right) = 0.0634 \quad \Rightarrow \quad \varphi'' = -1.25 * 10^{-9} \ Radians/mm$$

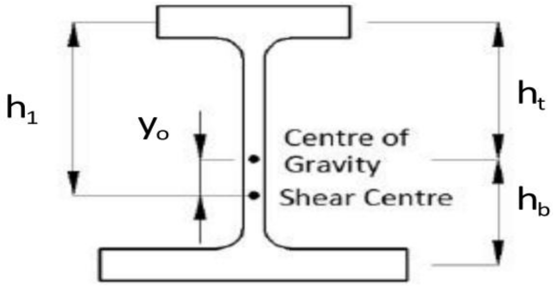
Interaction of top flange is critical due to beam asymmetry

Minor axis bending moment in top flange, M_{zT}:

$$M_{zT} = M_y \varphi = 128 * 0.0124 = 1.59 KNm$$

Minor axis bending stress in top flange, $\sigma_{bzT:} = \frac{M_{zT}}{\left(\frac{I_z}{0.5B_t}\right)} = \frac{1.59*10^3}{\left(\frac{10{,}463*10^{-5}}{0.50*183}\right)} = 1{,}390{,}471 Pa = 1.39 MPa$

Distance from center of gravity to shear center, y_0:



$$y_0 = \frac{h_b I_{yt} + h_t I_{yc}}{I_{yt} + I_{yc}} \qquad From\ Eqn. (4.7) of\ this\ thesis$$

$$h_t = y_e - \frac{t_f}{2} = 198 - \frac{40}{2} = 178mm$$

$$h_b = D - h_t - t_f = 342 - 178 - 40 = 124mm$$

$$I_{ZC} = \frac{t_f B_t{}^3}{12} = \frac{40 * 183^3}{12} * 10^{-4} = 2043cm^4$$

$$I_{zt} = \frac{t_f B_b{}^3}{12} = \frac{40 * 293^3}{12} * 10^{-4} = 8385cm^4$$

Hence, $y_0 = \frac{124*8385-178*2043}{8385+2043} = 64.84mm$

$$h_1 = h_t + h_0 \ = 242.84mm$$

Normalized Warping function, W_{no} is given by

$$W_0 = \frac{B_t h_1}{2} = 183 * \frac{242.84}{2} = 22{,}219mm^2$$

Warping stress, $\sigma_w$can be calculated as

$$\boldsymbol{\sigma_w = E.W_{no}./\varphi"/ = (210 * 10^9 * 22,219 * (1.25 * 10^{-9}) = 5,832,487Pa = 5.83Mpa}$$

Equivalent uniform moment factor, m_{LT} = 0.925 for a non-destabilizing UDL

Finally, putting values into interaction formula for buckling:

$$\frac{128 * 0.925}{883} + \left\{\frac{1.39 + 5.83}{355}\right\}\left\{1 + \frac{0.5.128 * 0.925}{883}\right\} = \mathbf{0.156} \ \therefore \quad \boldsymbol{OK}$$

Check Local Capacity of compression flange in combined bending and torsion

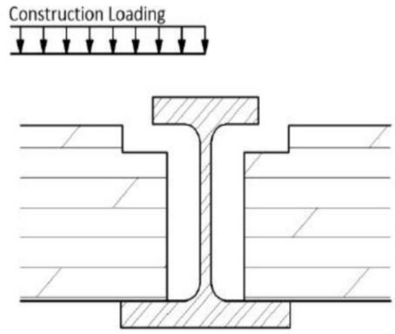
$\sigma_{by} + \sigma_{bzT} + \sigma_w \leq f_y$ (= 355 MPa)

$$\sigma_{by} = \frac{M_y}{W_{el,y}} = \frac{128 * 10^3}{2320} = 55.2MPa$$

$$\boldsymbol{\sigma_{by} + \sigma_{bzT} + \sigma_w \ = \ 55.2 + 1.39 + 5.83 \ = 62.40Mpa \qquad OK}$$

Construction Load Case 1 is OK for buckling and local capacity of compression flange

Case 2: XLT slab on both sides, construction loading on one side of ASB only.



Assume bearing length of 40mm for design

Imposed UDL, q_k	=	0.5 x 4.5m		= 2.25 kN/m
Permanent Load (slab)	=	Slab unit weight	= 4.7 x (260/1000)	= 1.22 kN/m2
Permanent Load (beam)				= 1.92 kN/m
Permanent UDL, g_k	=	slab + beam	= 1.22 x 9m) + 1.92	= 12.9 kN/m

$f_{d.1} = \gamma_G . g_k + \gamma_{Q1} . \psi_0 . q_k = (1.35 * 12.9) + (1.5 * 0.7 * 2.25) = \mathbf{19.80 KN/m}$

$f_{d.2} = \xi . \gamma_G . g_k + \gamma_{Q1} . q_k = (0.925 * 1.35 * 12.90) + (1.5 * 2.25) = \mathbf{19.50 KN/m}$

$f_d = \max(f_{d.1}, f_{d.2}) = \mathbf{19.80 KN/m}$

Lateral Torsional Buckling (LTB) for combined bending and tension

Verify against interaction formula:

$\frac{M_y m_{LT}}{M_{b,Rd}} + \left\{\frac{\sigma_{bzT} + \sigma_w}{f_y}\right\}\left\{1 + \frac{0.5.M_y m_{LT}}{M_{b,rd}}\right\} \leq 1$ From Equation (4.6) of this thesis

Major axis moment, M_y $= \frac{f_d l^2}{8} = \frac{19.80*9^2}{8} = 200KNm$

Torsion Load = 4.5m x 9m x (1.5 x 0.5) = 30.4 kN

Applied torsion, Tq = 30.4 x (126.5/103) = 3.85 kNm

Torsion Parameters:

As before,

L/ a = 9000/575 = 15.7

$\frac{\varphi G I_T}{T_q a}$ = 1.9 and= $\left(\frac{\varphi'' G I_T a}{T_q}\right)$ = 0.0634

Under new torsion loads and value of Tq,

$$\varphi = 4.40 * 10^{-3}\ Radians$$

$$\varphi'' = -4.45 * 10^{-10}\ Radians/mm^2$$

Minor axis bending moment in top flange, MzT:

MzT = $M_y \phi$ = 200 x 4.4 x10^{-3} = 0.88 kNm

Minor axis bending stress in top flange, σ_{bzT}:

$\sigma_{bzT:} = \frac{M_{zT}}{\left(\frac{I_z}{0.5B_t}\right)} = \frac{0.88*10^3}{\left(\frac{10,463*10^{-5}}{0.50*183}\right)} = 769{,}569Pa = 0.77MPa$

Warping stress, σ_w:

$\boldsymbol{\sigma_w = E.W_{no}./\varphi''/ = (210 * 10^9 * 22{,}219 * (4.45 * 10^{-10}) = 2{,}076{,}365Pa = 2.08Mpa}$

Interaction Formula for buckling:

$$\frac{200 * 0.925}{883} + \left\{\frac{0.77 + 2.08}{355}\right\}\left\{1 + \frac{0.5 * 200 * 0.925}{883}\right\} = \mathbf{0.218} \therefore \quad \boldsymbol{OK}$$

Check Local Capacity of compression flange in combined bending and torsion

$\sigma_{by} + \sigma_{bzT} + \sigma_w \leq f_y$ (= 355 MPa)

$$h_1 = h_t + h_0 \ = 242.84mm$$

Normalized Warping function, W_{no} is given by

$$W_0 = \frac{B_t h_1}{2} = 183 * \frac{242.84}{2} = 22{,}219mm^2$$

Warping stress, σ_w can be calculated as

$$\boldsymbol{\sigma_w = E.W_{no}./\varphi''/ = (210 * 10^9 * 22,219 * (1.25 * 10^{-9}) = 5,832,487Pa = 5.83Mpa}$$

Equivalent uniform moment factor, m_{LT} = 0.925 for a non-destabilizing UDL

Finally, putting values into interaction formula for buckling:

$$\frac{128 * 0.925}{883} + \left\{\frac{1.39 + 5.83}{355}\right\}\left\{1 + \frac{0.5.128 * 0.925}{883}\right\} = \mathbf{0.156} \ \therefore \quad \boldsymbol{OK}$$

Check Local Capacity of compression flange in combined bending and torsion

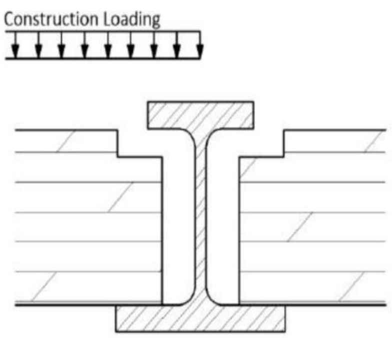
$\sigma_{by} + \sigma_{bzT} + \sigma_w \leq f_y$ (= 355 MPa)

$$\sigma_{by} = \frac{M_y}{W_{el,y}} = \frac{128 * 10^3}{2320} = 55.2MPa$$

$$\boldsymbol{\sigma_{by} + \sigma_{bzT} + \sigma_w \ = \ 55.2 + 1.39 + 5.83 \ = 62.40Mpa \qquad OK}$$

Construction Load Case 1 is OK for buckling and local capacity of compression flange

Case 2: XLT slab on both sides, construction loading on one side of ASB only.



Assume bearing length of 40mm for design

Imposed UDL, q_k	=	0.5 x 4.5m		= 2.25 kN/m
Permanent Load (slab)	=	Slab unit weight	= 4.7 x (260/1000)	= 1.22 kN/m2
Permanent Load (beam)				= 1.92 kN/m
Permanent UDL, g_k	=	slab + beam	= 1.22 x 9m) + 1.92	= 12.9 kN/m

$$\sigma_{by} = \frac{M_y}{W_{el,y}} = \frac{200 * 10^3}{2320} = 86.20MPa$$

$$\boldsymbol{\sigma_{by} + \sigma_{bzT} + \sigma_w = 86.20 + 0.77 + 2.08 = 89.10Mpa \quad OK}$$

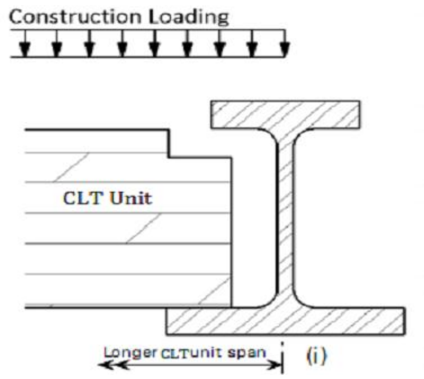
Construction Load Case 2 is OK for buckling and local capacity of compression flange

Construction Load case 3 is not critical as no torsion due to assumption of equal slab spans either side of ASB.

A.4.3 Normal Stage

Imposed Load = Office Loading = 2.5 kN/m2

Case 1: Pattern Loading, super-permanent loading on both sides, imposed loading on one side of ASB only.



Assume bearing length of 40mm for design

Imposed UDL, qk = 2.5 x 4.5m = 11.25 kN/m

Permanent Load (slab) = Slab unit weight = 4.7 x (260/1000) = 1.22 kN/m2

Permanent Load (beam) = 1.92 kN/m

Permanent UDL, gk = (slab + superimposed) + beam = (1.22+0.5)x9m + 1.92 = 17.4 kN/m

$$f_{d.1} = \gamma_G . g_k + \gamma_{Q1} . \psi_0 . q_k = (1.35 * 17.4) + (1.5 * 0.7 * 11.25) = \mathbf{35.30KN/m}$$

$$f_{d.2} = \xi . \gamma_G . g_k + \gamma_{Q1} . q_k = (0.925 * 1.35 * 17.4) + (1.5 * 11.25) = \mathbf{38.60KN/m}$$

$$f_d = \max(f_{d.1}, f_{d.2}) = \mathbf{38.60KN/m}$$

Lateral Torsional Buckling (LTB) for combined bending and tension

Verify against interaction formula:

$$\frac{M_y m_{LT}}{M_{b,Rd}} + \left\{\frac{\sigma_{bzT} + \sigma_w}{f_y}\right\}\left\{1 + \frac{0.5.M_y m_{LT}}{M_{b,rd}}\right\} \le 1$$

From Equation (4.6) of this thesis

Major axis moment, M_y $= \frac{f_d l^2}{8} = \frac{38.60 * 9^2}{8} = 391KNm$

Torsion Load = 4.5m x 9m x (1.5 x 2.5) = 152 kN

Applied torsion, Tq = 152 x (126.5/10^3) = 19.20 kNm

Torsion Parameters:

As before,

L/ a = 9000/575 = 15.7

$$\frac{\varphi G I_T}{T_q a} = 1.9 \ \& = \left(\frac{\varphi" G I_T a}{T_q}\right) = 0.0634$$

Under new torsion loads and value of Tq,

$$\varphi = 0.022\ Radians$$

$$\varphi" = -2.22 * 10^{-9}\ Radians/mm^2$$

Minor axis bending moment in top flange, MzT:

MzT = Myϕ = 391 x 0.022 = 8.60 kNm

Minor axis bending stress in top flange, σ_{bzT}:

$$\sigma_{bzT:} = \frac{M_{zT}}{\left(\frac{I_Z}{0.5B_t}\right)} = \frac{8.60*10^3}{\left(\frac{10,463*10^{-5}}{0.50*183}\right)} = 7{,}520{,}788Pa = \mathbf{7.52}\boldsymbol{MPa}$$

Warping stress, σ_{w}:

$$\boldsymbol{\sigma_w = E.W_{no}./\varphi"/ = (210 * 10^9 * 22,219 * (2.22 * 10^{-9}) = 10,358,498a = 10.40Mpa}$$

Interaction Formula for buckling:

$$\frac{391 * 0.925}{883} + \left\{\frac{7.52 + 10.40}{355}\right\}\left\{1 + \frac{0.5 * 391 * 0.925}{883}\right\} = \mathbf{0.47} \therefore \quad \boldsymbol{OK}$$

Check Local Capacity of compression flange in combined bending and torsion

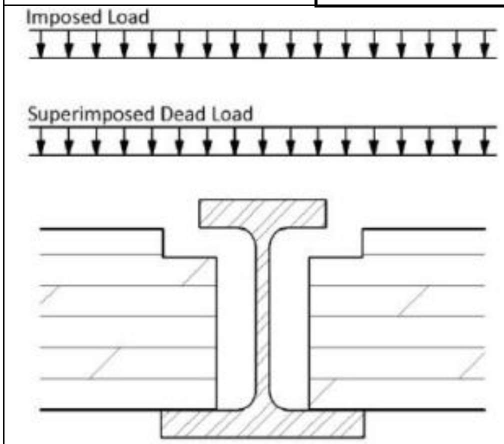
$\sigma_{by} + \sigma_{bzT} + \sigma_w \leq f_y$ (= 355 MPa)

$$\sigma_{by} = \frac{M_y}{W_{el,y}} = \frac{391 * 10^3}{2320} = 169MPa$$

$$\boldsymbol{\sigma_{by} + \sigma_{bzT} + \sigma_w = 169 + 10.40 + 7.52 = 187Mpa \quad OK}$$

Normal Stage pattern loading is OK for buckling and local capacity of compression flange

Case 2: Maximum loading



For this load case, need only check against buckling capacity and shear failure

Imposed UDL, qk = 2.5 x 9m = 22.5 kN/m

Permanent Load (slab) = Slab unit weight = 4.7 x (260/1000) = 1.22 kN/m2

Permanent Load (beam) = 1.92 kN/m

Permanent UDL, gk = (slab + superimposed) + beam = (1.22+0.5)x9m + 1.92 = 17.4 kN/m

$$f_{d.1} = \gamma_G . g_k + \gamma_{Q1} . \psi_0 . q_k = (1.35 * 17.4) + (1.5 * 0.7 * 22.50) = \mathbf{47.10 KN/m}$$

$$f_{d.2} = \xi . \gamma_G . g_k + \gamma_{Q1} . q_k = (0.925 * 1.35 * 17.4) + (1.5 * 22.50) = \mathbf{55.50 KN/m}$$

$$f_d = \max(f_{d.1}\, , \, f_{d.2}) = \mathbf{55.50 KN/m}$$

Lateral Torsional Buckling (LTB) for combined bending and tension

Verify against interaction formula:

$$\frac{M_y m_{LT}}{M_{b,Rd}} + \left\{\frac{\sigma_{bzT} + \sigma_w}{f_y}\right\}\left\{1 + \frac{0.5 . M_y m_{LT}}{M_{b,rd}}\right\} \le 1$$

From Equation (4.6) of this thesis

Major axis moment, M_y $= \frac{f_d l^2}{8} = \frac{38.60 * 9^2}{8} = 391 KNm$

Major axis moment, M_y $= \frac{f_d l^2}{8} = \frac{55.5 * 9^2}{8} = 562 KNm$

Buckling Capacity

$$\frac{M_y}{M_{b,Rd}} \le 1.00 \quad ; \quad \frac{562}{883} = \mathbf{0.636} \quad \therefore \ \boldsymbol{ok}$$

Shear Capacity

Design shear force, V_d $= \frac{f_d . l}{2} = \frac{55.5 * 9}{2} = 250 KN$

Design shear resistance $V_{pl,Rd}$ $= \frac{f_{y/\sqrt{3}}}{\gamma_{M0}}$

$$A_v = A - \left(B_b t_f + B_t t_f\right) + (t_w + 2r) t_f$$

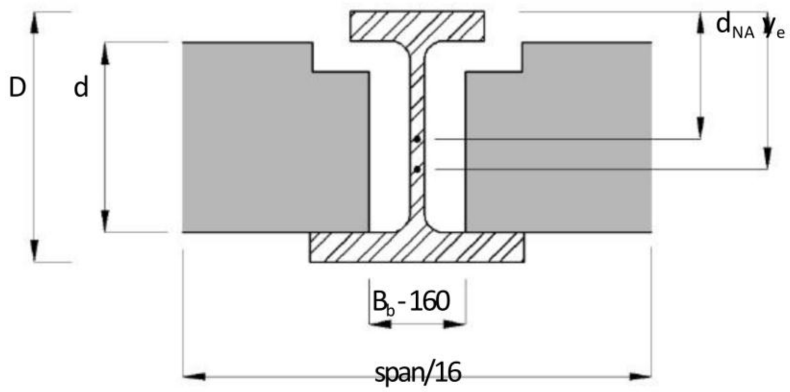
$$= (249 * 100 - (293 * 40 + 183 * 40) + (20 + 227) * 40 = 8820 mm^2$$

$V_{pl,Rd} \quad = 8820 * \frac{\left(\frac{355}{\sqrt{3}}\right)}{1.0} \quad = \quad 1{,}807{,}741N = \mathbf{1{,}807}\boldsymbol{KN}$

$V_d / V_{pl,Rd} \; \frac{Vd}{Vpl,Rd} \leq 1.00 \; = \frac{250}{1807} \; = \mathbf{0.138} \; \therefore \; \mathbf{Ok}$

A.4.4 Serviceability Checks

Composite behavior of the timber slab and steel beam is assumed for deflection checks at the normal loading stage. The 2nd moment of area of the composite beam section must be found for use in the calculations.



Effective Breadth

When using concrete slabs, with infill concrete around the web of the beam, effective breadth is assumed as span/32 to either side of the beam. To replicate this for the timber case, without infill, the effective breadth, b_{eff} is taken as

$b_{eff} \; = \frac{span}{16} - (B_b - 160mm) \; = \; \frac{9000}{16} - (293 - 160mm) = 430mm$

This accounts for the area between the slab and the ASB web being empty.

Modular Ratio

The modular ratio, n, is a ratio of the Young's Moduli of the two materials in the composite section and is used to make their different properties compatible in calculations. By using the modular ratio it is possible to transform the cross-section area of one material into an equivalent area of the other material. This additional equivalent area is used to calculate the enhancement in behavior of the two materials acting compositely versus the one material acting alone. To transform the timber slab into an equivalent area of steel, the modular ratio is given by,

$$n = \frac{E_{steel}}{E_{timber}} \quad , \quad \textit{From Equation (4.17)of this thesis}$$

Due to its susceptibility to creep, the deflection of timber elements normally consists of two parts - an instantaneous deflection, and an additional long term deformation. However, due to the presence of a substantially stiffer element, the ASB, and the use of composite theory, it is thought sufficient to account for loss of stiffness of the timber over time when calculating the Young's Modulus.

$$As\ such,\ \ E_{timber} = E_{d,SLS} = \frac{E_{mean}}{(1+k_{def})} = \frac{12GPa}{1+0.8} = 6.67GP\ ,\quad n = \frac{210}{6.67} = 31.50$$

1st Moment of Area to find composite neutral axis depth, y_{comp}

About the top of the slab, in steel units...

$$\left[\left(\frac{b_{eff}.d}{n}\right) + A_{ASB}\right].y_{comp} = \left(\frac{b_{eff}.d}{n}\frac{d}{2}\right) + (A_{ASB}.d_{NA})$$

Where AASB is the cross sectional area of the ASB, and d_{NA} is the distance between the top of the slab and the neutral axis of the ASB, and is given by:

$$d_{NA} = y_e - \left(D - t_f - d\right) = 198 - (342 - 40 - 260) = 156mm$$

$$\left[\left(\frac{430*260}{31.50}\right) + (249*10^2\right].y_{comp} = \left(\frac{430*260^2}{2*31.50}\right) + (249*10^2)*156$$

$$\boldsymbol{y_{comp} = 152.80mm}$$

Composite 2nd Moment of Area, using Parallel Axis Theorem

In steel units...

$$I_{comp} = \left[\left\{\left(\frac{b_{eff}.d^3}{12n} + \{\frac{b_{eff}.d}{n} * (y_{comp} - \frac{d}{2}\right)^2\right\}\right] + [I_{ASB} + A_{ASB}.(d_{NA} - y_{comp})^2]$$

Where IASB is the 2nd moment of area for the ASB. Substituting values into the formula,

$$\mathbf{I_{comp} = 48{,}078\ cm^4}$$

Construction stage deflection check (bare steel properties)

Self-weight deflection (slab on both sides):

$$\omega_{SLS} = slab + beam = (1.22*9) + 1.92 = 12.90KN/m$$

Deflection during construction $\delta_c, is\ given\ by,$

$$\delta_c = \frac{5}{384}\frac{\omega_{SLS}L^4}{EI} = \frac{5}{384}\frac{12.90*10^3*9^4}{210*48{,}078*10} = 11.40mm$$

$$\text{Construction Deflection Limit} = \frac{span}{200} = \frac{9000}{200} = 45.00mm\ \ \boldsymbol{OK}$$

formal stage deflection check (composite properties)

Imposed load deflection check

$$\omega_{SLS} = \quad Impossed\ load \quad = \quad 2.5 * 9.00m \quad = \quad 22.50KN/m$$

$$\delta_{IMP} = \frac{5}{384}\frac{\omega_{SLS}L^4}{EI} = \frac{5}{384}\frac{22.5 * 10^3 * 9^4}{210 * 48{,}078 * 10} = 19.00mm$$

$$Impossed\ Diflection\ limit = \frac{span}{360} = \frac{9000}{360} = 25.00mm \quad \boldsymbol{OK}$$

Super-imposed permanent load deflection check

$$\omega_{SLS} = \quad Impossed\ load \quad = \quad 0.5 * 9.00m \quad = \quad 4.50KN/m$$

$$\delta_{SUP} = \frac{5}{384}\frac{\omega_{SLS}L^4}{EI} = \frac{5}{384}\frac{4.5 * 10^3 * 9^4}{210 * 48{,}078 * 10} = 3.81mm$$

$$Impossed\ Diflection\ limit = \frac{span}{360} = \frac{9000}{360} = 25.00mm \quad \boldsymbol{OK}$$

As deflections arising during construction are corrected on site, total deflection consists solely of those due to imposed load and superimposed permanent load:

$$Total\ Deflection = \delta_{IMP} + \delta_{SUP} = 19.00mm + 3.81mm = 22.80mm \quad \boldsymbol{OK}$$

$$Check\ 22.80mm < \frac{Span}{200}\ i.e \frac{9000}{200} = 45mm \quad Hence\ \boldsymbol{OK}$$

300 ASB 196 is verified for this situation and loading

A.4.5 Tie Beam

Assuming an internal bay,

Design Tie force, Ti = $0.8(g_k + \psi_1 q_k)sL$

S is the tie spacing = 9m

L is the tie span = 9m

ψ_1 = 0.5

g_k slab weight + superimposed = 1.72 kN/m2

q_k imposed load = 2.5 kN/m2

Substituting, Ti = 192 kN

Minimum section area required= $\frac{T_i}{f_y} = \frac{192}{355} * 10 = 5.41cm^2$

To avoid visible sag, min. section depth = span/40 = 9000/40 = 225mm

To meet requirements, use 165x267x33 T-section as structural tie.